ENGINEERING DIVISION WORKING COPY
RETURN TO FILE

Mr. Brown

MERRIMACK VALLEY FLOOD CONTROL

DEFINITE PROJECT REPORT FOR HOPKINTON-EVERETT RESERVOIR

CONTOOCOOK AND PISCATAQUOG RIVERS

(REVISED, SEPTEMBER 27, 1940)

CORPS OF ENGINEERS, U.S. ARMY

U.S. ENGINEER OFFICE BOSTON, MASS.

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
3D FLOOR, PARK SQUARE BLDG.
31 ST. JAMES AVENUE
BOSTON, MASS.

September 27, 1940

Subject: Revised Definite Project Report for Hopkinton-Everett Reservoir - Merrimack River Basin

To: The Chief of Engineers, U.S. Army, Through the Division Engineer, North Atlantic Division, New York, N.Y.

1. Project Authority.- The Hopkinton-Everett Reservoir described herein is proposed as an element of the comprehensive plan for flood control and other purposes for the Merrimack River Basin, authorized by the Flood Control Acts approved June 22, 1936 and June 28, 1938, and as described in the Annual Report of the Chief of Engineers for 1940.

2. Previous Investigations.- (a) The investigations and studies upon which the authorized project for the Merrimack River Basin is based are described in House Document No. 689, 75th Congress, 3d Session. A comprehensive plan of flood control reservoirs and related flood control works, at an estimated total cost of $21,000,000, was recommended in that document. A preliminary definite project report recommending Hopkinton-Everett Reservoir as an element of the comprehensive plan was submitted on September 16, 1939, (E.D. 7402(Merrimack R., Contoocook Diversion)1). This report was approved on October 14, 1939, to the extent of authorizing the continuation of field investigation. A final definite project report was submitted on February 1, 1940, (E.D. 7402 (Merrimack R., Hopkinton-Everett Res.)-8), and approved on March 12, 1940, subject to further consideration of the outlet capacity for Everett Dam. Prior to final design of the Hopkinton-Everett project, the consent of the State of New Hampshire to acquisition of land was requested on March 13, 1940. In the course of a series of hearings held by the State on this matter in May 1940, the only major objection to the project expressed by local interests was that it required the removal of the Davis Paper Co. and the village of West Hopkinton. Representatives of this office explained that studies were then under way for a revised lay-out which would make unnecessary the removal of the paper company and the village. The objectors indicated that the plan would then be satisfactory to them.

(b) During the same hearings referred to above, the Federal Power Commission formally proposed a system of reservoirs in the Contoocook River Basin as an alternate to the proposed Hopkinton-Everett Reservoir. The Department thereupon undertook detailed investigation of the Commission's proposal. A report on the results of these studies, together with an analysis of the respective merits of the Commission's plan and Hopkinton-Everett Reservoir, was submitted on August 12, 1940, (E.D. 7497-213). The report, which concluded that the Hopkinton-Everett project and other related projects previously selected by the Chief of Engineers for the comprehensive plan were superior to the Commission's plan, was approved on August 26, 1940.

3. Scope of This Report.- A revision of the definite project report for Hopkinton-Everett is submitted at this time, (1) to take into account the further consideration of the outlet capacity for Everett Dam referred to above, and (2) to present for approval a revised lay-out of structures which will reduce the extent of disruption of local affairs necessary for the project. The general plan of the project has not been altered since the approved February 1 report. The approved spillway design criteria have not been changed, and the reservoir capacity, total cost and ratio of benefits to costs have been only slightly modified as a result of the revisions herein.

4. Definite Project Plan.- (a) Relation to Other Flood Control Projects.- Data on the proposed Hopkinton-Everett Reservoir and the related projects selected by the Chief of Engineers under the general Merrimack Basin authorization (paragraph 1) are as follows (see Plate 1):

Project	Drainage Area (Sq.Mi.)	Storage Capacity (Ac.-Ft.)	Estimated Total Cost	Status
Franklin Falls	1000	170,000	$ 7,883,000	Under construction
Blackwater	128	46,000	1,300,000	Under construction
West Peterboro	44	16,000	1,011,000	Def.proj. approved
Mountain Brook	14	4,800	344,000	Def.proj. approved
Hopkinton-Everett	432	157,000	11,300,000	-
Total	1618	393,800	$21,838,000	-

(b) Location and Description.- The proposed Hopkinton-Everett Reservoir is located within the towns of Henniker, Hopkinton, Weare and Dunbarton in Hillsboro and Merrimack Counties, Merrimack River Basin, New Hampshire, (see Plates 1 and 2). The proposed reservoir will be formed by a dam, a canal (No. 1), and two dikes (H-2 and H-3) in the Contoocook River Basin upstream from the village of West Hopkinton, and a dam and two dikes (P-1 and P-2) in the Piscataquog River Basin near the Everett railroad station (no longer used). The separate storage areas thus formed in the Contoocook and Piscataquog River basins will be connected by a canal (No. 2) of sufficient capacity to cause the storage areas to function as a single reservoir. The combination reservoir will not interfere with the low-water flows of either the Contoocook or Piscataquog Rivers. Stream flow conditions at all points below the project will not be adversely affected at any time and will be materially benefitted during flood periods.

(c) Reservoir Capacity.- The reservoir design flood has a volume equivalent to about 11.5 inches of run-off and is similar in magnitude and manner of occurrence to the flood of March 1936, the maximum of record. Based on the relation of corresponding storage and discharge capacities required to control the reservoir design flood, a total reservoir capacity of 157,000 acre-feet (6.0 inches) and a total reservoir outlet design discharge of 8,000 second-feet were selected. This capacity is obtained at pool elevation 412, which was adopted for the West Hopkinton spillway crest elevation. The spillway at Everett Dam was set at El. 414 in order to limit the frequency and amount of spillway discharge on the Piscataquog River and thus preserve a balance with respect to the channel capacities of the two rivers involved. Similar considerations governed the selection of the design discharges for the outlets at the two dams. The maximum capacity of the reservoir, based on control of the gross drainage area of 490 square miles, is equivalent to about 6 inches of run-off. The proposed projects of Mountain Brook and West Peterboro, which are upstream from Hopkinton-Everett and which reduce the drainage area to be controlled by Hopkinton-Everett to 432 square miles, are small projects designed primarily for local control. They have little practical effect on the design criteria for Hopkinton-Everett Reservoir, principally because of the method of operation proposed wherein the outlets at the West Hopkinton Dam will be shut off entirely for a portion of the flood period in order to obtain the maximum desynchronization of the Contoocook River flood peak at its junction with the main stream of the Merrimack and thus obtain the maximum flood reduction effect at the principal damage centers on the Merrimack. Furthermore, no economic advantage is gained by adopting a lesser storage capacity than 6 inches over the 490 square miles for Hopkinton-Everett because the costs of the lesser storage are not proportionately less. This is illustrated by the cost curve on Plate 26. The cost of storage in the range from 5.1 to 6 inches is practically constant, due to the fact that, for lesser storage with its corresponding lesser cost for dams and dikes, there is an increased cost for deeper canals. Details of the reservoir design and hydraulic factors are given in Appendixes A and B.

(d) Conservation Storage and Power Development Possibilities.- The cost of obtaining additional storage above the adopted capacity of 157,000 acre-feet is excessive, as illustrated by the abrupt upward sloping of the cost curve on Plate 26. This sharp increase in cost for additional capacity is due principally to the excessive costs for land, relocations and rights-of-way which would result in the town of West Henniker at the upstream end of the Contoocook River portion of the reservoir as now proposed. The prospective benefits of conservation storage use and power development are not sufficient to warrant the excessive costs of additional storage. If desired at some future date, the proposed Hopkinton-Everett Reservoir could be utilized for multiple purposes by reallocation of the storage capacity selected at this time for flood control alone. Such multiple-purpose use would be feasible if additional flood control and conservation storage were developed upstream from Hopkinton-Everett in such amount as to permit utilization of about 30,000 acre-feet of storage in the Hopkinton (Contoocook River) section of the reservoir for conservation purposes. If this were done, it would be feasible to install 5,000 kw. of generating capacity for a primary load factor demand of approximately 20 per cent. The present design of structures is such that no provisions are necessary at this time for the utilization of the reservoir for conservation purposes in the future. Also, no provisions are necessary for the possible future power installation, since this can be accomplished at any time in the future at a site near the proposed spillway adjacent to Dike H-3.

(e) Time Required to Empty Reservoir.- Sufficient outlet capacity will be provided in both dams to double the outlet design discharge for emptying purposes. With this capacity the West Hopkinton section can be emptied from full pool in 6 days and the Everett section emptied in 12 days. Three inches, or one-half of the total storage of the reservoir, will be recovered in 3 days. Five inches, or 83% of the total storage, will be recovered in less than 7 days.

(f) Spillway Requirements.- Spillways will be provided at both dams with a total capacity sufficient to pass a spillway design flood with a total volume of 17.7 inches and a peak inflow of 137,500 second-feet, which is about four times the estimated maximum inflow of record for the drainage area controlled. The peak inflow of the spillway design flood has a coefficient "C" of 6,220 in the relation, Peak Discharge $= C \sqrt{\text{Drainage Area}}$. Freeboard of from 5.5 to 6.7 feet is provided for the various dams and dikes involved, based on the requirements for wave action, ride-up of waves, and wind set-up as outlined in Engineer Bulletin R. & H. No. 9, 1938. Details of the spillway requirements are contained in Appendix A.

5. Description of Structures and Work Involved.- (a) Reservoir Area.- The reservoir area at El. 412 is 6,300 acres, of which approximately 66% is wooded, 14% is in pasture, and 20% is tillable. Only one small center of population, the village of East Weare, involving less than 90 persons, is included in the reservoir area under the revised lay-out. The village of West Hopkinton, which was affected under the lay-out originally considered, will not be disturbed with the project as now proposed. The project will involve the relocation of approximately 21 miles of roads and highways, 5.25 miles of railroad, and 2,880 graves distributed in 5 cemeteries. The various structures required for the reservoir may be divided into three general groups in which the construction operations are related: (1) West Hopkinton structures, including West Hopkinton Dam, Dikes H-2 and H-3, Canal No. 1, and the spillway; (2) Everett Dam; (3) Canal No. 2 and Dikes P-1 and P-2. Geology, soil and design data for all structures as described in the following paragraphs are given in Appendix C. A detailed estimate of cost is given in Appendix D.

(b) West Hopkinton Dam.- The location formerly proposed for this dam was about 1/2 mile downstream from the village of West Hopkinton on the Contoocook River. The site now proposed is about 1/4 mile upstream from the village of West Hopkinton. The structure will consist of an earth embankment with a length of 670 feet, maximum height of 75 feet, and a gross volume of 240,000 cubic yards. The dam will contain two reinforced concrete outlet conduits. Each conduit will be 11 feet wide by 13 feet high and will be controlled by two 7'x10' gates.

(c) Canal No. 1 will be located just upstream from West Hopkinton Dam and will consist of a channel cut through the ridge separating the Contoocook River and Elm Brook valleys. The maximum depth of cut will be about 110 feet. The total length of the canal will be 2,050 feet, with a bottom width varying from 130 to 150 feet for a length of 1,130 feet, which constitutes the principal section of the canal. The excavation from Canal No. 1 will be utilized for fill in West Hopkinton Dam and Dikes H-2 and H-3.

(d) Dike H-2 is located on Elm Brook above its junction with the Contoocook River. The dike will have a total length of 3,600 feet, maximum height of 70 feet, and a gross volume of 986,000 cubic yards of rolled earth and rock fill.

(e) Dike H-3 will be located between the Elm Brook and Contoocook River valleys. The proposed structure will consist of an embankment with a total length of 3,900 feet, maximum height of 60 feet, and a gross volume of 840,000 cubic yards of rolled earth and rock fill.

(f) West Hopkinton spillway will be located adjacent to Dike H-3 and will consist of a channel 300 feet wide with a concrete weir 650 feet long founded on bedrock. The earth excavation for the spillway will be utilized for Dike H-3. Rock excavation will be used for West Hopkinton Dam, Dikes H-2 and H-3, and for the floor and side slopes of Canal No. 1.

(g) Everett Dam.- The site for the proposed dam is located on the Piscataquog River approximately 1-1/2 miles southeast of the village of East Weare, New Hampshire. Everett Dam will consist of an earth embankment 1,250 feet long with a maximum height of 115 feet and a gross volume of approximately 1,000,000 cubic yards of rolled earth and rock fill. The outlet works will consist of two rectangular conduits 7 feet wide by 8 feet 6 inches high, each controlled by a 7- by 9-foot gate. The spillway will be of concrete, 212 feet long, and founded on bedrock throughout.

(h) Canal No. 2 will connect the valleys of Elm Brook (Contoocook River Basin) and Choate Brook (Piscataquog River Basin). The proposed canal will have a total length of about 14,500 feet, a bottom width of 170 feet, and will require the excavation of 1,500,000 cubic yards of earth and 84,000 cubic yards of rock. Excavation materials will be utilized in Dikes P-1 and P-2.

(i) Dikes P-1 and P-2 will be located near the northeast extremity of the Everett section of the reservoir. Dike P-1 will have a top length of 3,550 feet, maximum height of 48 feet, and a gross volume of 480,000 cubic yards of fill. Dike P-2 will have a top length of 2,350 feet, maximum height of 27 feet, and gross volume of 120,000 cubic yards.

(j) Time required for construction will be from two to four years, depending on the rate of progress desired (see Appendix C, Part V, "Construction Procedure"). It is estimated that construction operations can be started about six months after the date of approval of this revised definite project report, assuming consent to the acquisition of lands for the project has been obtained from the State of New Hampshire (see paragraph 8).

6. Estimated Cost.- A detailed estimate of cost is given in Appendix D. The estimated initial cost is summarized as follows:

Land, rights-of-way and relocations . . .	$ 3,325,000
Construction (including 2 dams, 4 dikes and 2 canals)	7,975,000
Total	$11,300,000

The annual carrying charges computed on the basis of 3-1/2% interest on the investment, with amortization of fixed parts in 50 years and of movable parts in 25 years, and including an allowance for operation and maintenance of $10,000, amount to $517,000. The foregoing costs include allowance for relocation of railroads affected by the reservoir. The railroad in question, however, is operating for freight traffic only at this time. It is possible that abandonment of the line may be obtained and arrangements made to provide freight service from another direction at less cost than necessary for relocation of the present line. If this should occur, the estimated cost will be $700,000 less than indicated above.

7. Economic Analysis.- Hopkinton-Everett Reservoir is proposed as part of the authorized comprehensive reservoir system for the Merrimack River Basin. The proposed project, together with the other reservoirs selected for the plan, will control about 32% of the total Merrimack River Basin area and will afford flood control benefits amounting to $1,143,000 annually, or nearly three-fourths of the total estimated average annual flood losses in the basin. Control of the Contoocook River is essential for a satisfactory comprehensive flood control plan of the Merrimack River Basin. The proposed Hopkinton-Everett Reservoir has been found to be the most practicable project for this purpose. The overall ratio of benefits to costs of the comprehensive plan, including Hopkinton-Everett, is 1.13.

8. Local Cooperation.- (a) Desires of Local Interests.- As mentioned in paragraph 2, consent of the State of New Hampshire to acquisition of lands for the project, as originally proposed, was requested on March 13, 1940. At a series of hearings held by the State in this connection, no objections were received from the local interests in the vicinity of East Weare, one of the communities affected by the plan. Residents of the town of Hopkinton, however, objected to the proposed removal of the Davis Paper Company, their principal industry, and the disturbing of the village of West Hopkinton. It was brought out at the hearings, however, that local interests would have no objection to the project if the Davis Paper Company and the village of West Hopkinton were not affected. Since the revised lay-out of structures proposed at this time accomplishes this objective, it is believed that there will be no serious objection to the project on the part of local interests and that consent of the State to the acquisition of lands will be forthcoming.

(b) Local Contribution Required.- Since this project is a reservoir affording general flood control benefit and is proposed for construction under a general basin project authorized by the Flood Control Act of 1938, no local financial contribution is required.

9. <u>Recommendation</u>.- It is recommended that the foregoing revised definite project plan, which provides additional outlet capacity for Everett Dam and which includes a revised lay-out of structures involving less disruption of local affairs than previously required, be adopted.

L. B. Gallagher
Lieut. Col., Corps of Engineers
District Engineer

Appendixes:
- A - Hydrology Report
- B - Hydraulics
- C - Geology, Soil and Design Data
- D - Detailed Estimate of Cost
- E - Illustrations

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX A

HYDROLOGY REPORT

Table of Contents

List of Tables

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX A

HYDROLOGY REPORT

1. Scope.- The data contained in this appendix constitute a report on the hydrology of the Contoocook and Piscataquog River Basins as affecting the spillway design flood and spillway requirements for Hopkinton-Everett Reservoir in accordance with the procedure outlined in Engineer Bulletin R. & H. No. 9, 1938. In addition, the report contains the data and procedure used in determining the reservoir design flood. The data herein were originally studied on the basis of the project controlling 490 square miles of drainage area. Since the original computations, two additional reservoirs (Mountain Brook and West Peterboro, see Plate 1) controlling 58 square miles upstream from Hopkinton-Everett have been proposed. The gross figure of 490 square miles has nevertheless been retained for computing spillway and reservoir capacity requirements as a measure of conservatism, since both upstream projects are small and designed primarily for local control. Wherever pertinent, the effects of the upstream projects are mentioned for comparison with the adopted basis of analysis.

2. Project Description.- (a) General.- Hopkinton-Everett Reservoir is formed by a dam, a canal (No. 1), and 2 dikes (H-2 and H-3) on the Contoocook River near West Hopkinton and a dam and 2 dikes (P-1 and P-2) on the Piscataquog River near Everett. The separate storage areas thus formed are connected by a canal (No. 2) of sufficient capacity to cause them to act as a single reservoir during floods. (See Plates 1, 2 and 38.) The total storage capacity is 157,000 acre-feet, amounting to 6.0 inches over the 490 square miles of drainage area controlled. Area and capacity curves are shown on Plate 3.

(b) West Hopkinton Section.- The dam site on the Contoocook River is 1/4 mile above the village of West Hopkinton, N.H., and 17.3 miles above the junction of the Contoocook and Merrimack Rivers. The tributary drainage area above the dam is 426 square miles. With spillway crest at El. 412, the storage is 60,500 acre-feet, covering an area of 3,300 acres. The storage capacity of 60,500 acre-feet is equivalent to 2.7 inches of run-off from the 426-square-mile drainage area. The formation of the West Hopkinton section of the reservoir requires the construction of (1) the dam on the Contoocook River that will include the outlets, (2) a canal (No. 1) to divert water from the river to the Elm Brook pool, (3) two dikes (H-2 and H-3), (4) a spillway located near the dike H-3 of sufficient capacity to pass a spillway design flood from the Contoocook River, and (5) a small drainage ditch to drain the area near the spillway. For purposes of identification, the West Hopkinton section of the reservoir is subdivided into three pools: (1) the Contoocook River pool, which extends from Henniker to the West Hopkinton Dam and Canal No. 1; (2) the Elm Brook pool, which has the spillway and extends from Canal No. 1 to Canal No. 2; and (3) the spillway pool, which includes the reservoir area contiguous to the spillway and Dike H-3. The elevation of the water surface in this pool gives the head used in computation of the spillway rating curve.

(c) Everett Section.- The dam site on the North Branch Piscataquog River is near the Everett railroad station (no longer used) in the town of East Weare, N.H., and 16 miles above the junction of the Piscataquog and Merrimack Rivers. This dam contains the two conduit outlets and spillway. In addition, two dikes are required at low sections in the rim of the reservoir area. The tributary drainage area above the dam is 64 square miles. At pool elevation 412, the storage is 96,500 acre-feet, covering an area of 3,000 acres. The storage capacity is equivalent to 28.3 inches of run-off from the 64-square-mile drainage area.

(d) Canal No. 2.- A connecting canal, about 2 miles long, with invert at El. 385, makes possible the utilization of the large amount of storage available on the Piscataquog River to make up the deficiency of storage on the Contoocook River. The combination reservoir will not interfere with the low-water flows of either river but will provide adequate storage (6.0 inches) to control flood flows over the total drainage area of 490 square miles.

3. Basin Characteristics.- (a) Contoocook River.- The drainage area of the Contoocook River is shown on Plate 1. Profiles of the main stream and principal tributaries are shown on Plate 4. The Contoocook River has a length of about 50 miles above the West Hopkinton dam site (Mile 118.9), and the drainage area has an average length of 35 miles and width of 12 miles. The upper basin above Hillsboro (Mile 133.6) has a length about twice its width and is surrounded by ridges rising from 1200 to 1500 feet above mean sea level, with isolated peaks exceeding 2000 feet. Slopes and drainage areas at various points on the Contoocook River are listed in Table 1.

TABLE 1 - SLOPES AND DRAINAGE AREAS, CONTOOCOOK RIVER

Location*	Drainage Area (Sq.Mi.)	River Mile above Newburyport Light	Intermediate Distance in Miles	Elevation in Feet M.S.L.	Difference in Elevation Feet	Average Slope Feet Per Mile
Contoocook Lake	15	167.8		1003.5		
			2.6		45.6	17.5
E. Jaffrey	35	165.2		957.9		
			8.4		263.6	31.4
North Village	127	156.8		694.3		
			6.0		32.0	5.3
Cavendor	168	150.8		662.3		
			17.2		89.3	5.2
Hillsboro	349	133.6		573.0		
			4.1		38.8	9.5
W.Henniker Dam	368	129.5		534.2		
			.8		48.2	60.3
W.Henniker Gaging Station	368	128.7		486.0		
			9.8		109.0	11.1
W.Hopkinton Dam Site	426**	118.9		377.0		
			.2		17.6	88.0
W.Hopkinton	418	118.7		359.4		
			6.2		12.6	2.0
Above Warner River	440	112.5		346.8		
			.1		.2	2.0
Mouth Warner River	590	112.4		346.6		
			2.7		1.6	.6
Above Blackwater River	592	109.7		345.0		
			.1		.2	2.0
Mouth Blackwater River	728	109.6		344.8		
			4.1		1.8	.4
Riverhill Dam Site	756	105.5		343.0		
			4.2		81.5	19.6
Penacook Gaging Station	766	101.3		261.5		
			.5		18.0	32.7
Contoocook R. Mouth	767	100.8		243.5		

* See map, Plate 1 and profiles, Plate 4.

** Drainage area will include Elm Brook by construction of Dike H-2 and Canal No. 1.

From the West Hopkinton dam site to Hillsboro, 15.1 miles, the river is a series of flat reaches broken by one fall of over 100 feet in 2 miles and by low dams near West Henniker. There is only one tributary of appreciable size in this reach, namely, Amey Brook, entering from the left. Proceeding upstream, the 12-mile reach from Hillsboro to Bennington (Mile 146) has a slope of less than 2 feet per mile, with swampy areas and considerable valley storage. The North Branch of the Contoocook enters the main stream in this reach at Mile 135.1. There are a number of natural lakes and ponds, partially controlled by dams, on the North Branch and its tributaries. In the 12-mile reach between Bennington and Peterboro the river channel has a flat slope and wide valleys. The 20 miles of the Contoocook River valley above West Hopkinton dam site provide a large amount of valley storage that reduces flood peak flows appreciably. The drainage area to the right of the river has steep slopes with short travel to the main river, run-off from this area passing on before arrival of flow from the left portion of the drainage area. On the left portion of the drainage area the time of travel, and, therefore, time of concentration, is increased.

(b) Contoocook River Tributaries.- (See Table 2.) All of the large tributaries enter the Contoocook River from the left, and above West Hopkinton about 70 per cent of the drainage area is on the left side of the river. The effect of this characteristic is also to flatten the West Hopkinton hydrograph, giving a more prolonged but lower peak. Hydrographs for minor floods obtained at Hillsboro show flat and prolonged peaks. There are a large number of low-head mill dams and power dams on the tributaries and main stream of the Contoocook which, however, have little effective storage. Many of these dams were destroyed or badly damaged during the 1936 and 1938 floods. None of the existing reservoirs in the basin are large enough to affect materially the flood flow at West Hopkinton should it fail during a flood of the magnitude used for spillway design purposes.

(c) Piscataquog River.- The drainage basin of the Piscataquog River above the proposed Everett dam site is approximately 15 miles long and 4 miles wide. The topography of the basin consists of rolling hills conducive to a rapid run-off. River valleys have steep hydraulic gradients with no effective channel storage. There are two storage reservoirs in the tributary drainage area above Everett dam site. They are maintained at full pool whenever possible and, therefore, cannot be considered for possible flood control effect. The capacity of the reservoirs is small in comparison with the magnitude of the spillway design flood for Everett Dam and their failure would not appreciably affect the spillway requirements for Everett Dam.

TABLE 2 - CONTOOCOOK RIVER TRIBUTARIES
(WITH DRAINAGE AREAS EXCEEDING 10 SQUARE MILES)

Tributary*	Enters From:	Drainage Area at Mouth (sq.mi.)	Mileage at Confluence with Contoocook River (zero mileage at Newburyport Light)
Contoocook Lake	--	15	167.8
Mountain Brook	Left	14	167.7
Gridley River	Right	12	162.3
Meadow Brook	Right	12	161.6
Nubanusit Brook	Left	49	158.7
Boglie Brook	Right	13	155.4
Otter Brook	Right	16	153.6
Ferguson Brook	Left	12	151.9
Moose Brook	Left	14	149.4
Great Brook	Left	10	143.6
North Branch Contoocook	Left	121	135.1
Sand Brook	Left	10	130.2
Amey Brook	Left	21	122.6
Elm Brook	Right	10	117.8
Warner River	Left	150	112.4
Blackwater River	Left	136	109.6
Deer Meadow Brook	Left	19	109.3
Total		634	

* See map, Plate 1 and profiles, Plate 4.

4. Precipitation Stations and Records.- The location of precipitation stations in and adjacent to the Contoocook and Piscataquog Basins is shown on Plate 5. Data from stations used in analysis of the September 1938 storm are shown on Plate 6. The only recording gage in the immediate vicinity is at Concord, N.H., about 10 miles from the West Hopkinton dam site. The mass curves of precipitation for stations having only one or two observations daily were drawn to correspond with the Concord curve.

5. Stream Flow Data.- (a) Contoocook River. The U.S. Geological Survey maintains three stream gaging stations in the Contoocook River Basin (see Plate 5): (1) Contoocook River at Penacook, N.H., 0.7 mile above the mouth of the river, with a drainage area of 766 square miles; (2) Blackwater River near Webster, with a drainage area of 129 square miles, and (3) North Branch of the Contoocook River near Antrim, 6 miles above its confluence with the Contoocook River, with a drainage area of 54.8 square miles. Length of records ranges from 10 to 20 years, but of these three stations only Antrim lies within the proposed reservoir drainage basin. Gage readings were recorded at the New Hampshire Public Service Company dam at West Henniker (drainage area 368 square miles) and were used in deriving the 1938 flood hydrograph. This flood was analyzed for distribution values and is shown on Plate 7. Peak flow values have been determined at mill and power dams for the 1938 and other floods and were useful as a check, but such data were insufficient to define an inflow hydrograph to the Hopkinton reach.

(b) Piscataquog River.- There are no stream flow data on the upper Piscataquog River suitable for obtaining distribution values. Stream flow records have been obtained from the New Hampshire Public Service Company for their Greggs Falls Dam, located near the mouth of the river (drainage area 201 square miles), but no accurate relationship can be derived from the hydrograph at this dam applicable to the 64-square mile drainage area above the Everett Dam.

6. Maximum Precipitation Values.- (a) Summer and Fall Conditions.- The rainfall data for determining the maximum summer storm were obtained from values furnished by the Hydro-Meteorological Section of the U. S. Weather Bureau, based on limiting precipitation values for one and 1000-square-mile areas for various durations. These were plotted on semi-log paper and connected by straight lines to obtain the depth-area curves shown on Plate 8. Data from these curves were compared with all data on maximum precipitation available in this office, including the September 1932 and October 1903 storms, and the selected data were found to give appreciably higher values of rainfall for the drainage areas being studied.

(b) Winter and Spring Conditions.- Maximum spring rainfall, snow run-off values, and minimum infiltration rate of 0.05 inch per hour, as obtained in the studies on the Pemigewasset River Basin (see Supplementary Hydrology Report, Franklin Falls Dam), were applied to the Contoocook and Piscataquog Basins. Depth-area curves for various durations of precipitation (see Plate 10) were obtained following the same procedure as for the summer-fall conditions. For the 1000-square-mile area, the maximum spring rainfall values as determined for the Pemigewasset Basin were plotted on a natural scale for 6, 12, 18, 24, 30, 36, 42, and 48-hour durations. For one square mile the maximum 48-hour spring rainfall recorded, that for Pinkham Notch, N.H., in March 1936, increased by 25 per cent, was used. The March 1936 storm was the largest spring storm of record, and the Pinkham Notch station recorded the greatest 24 and 48-hour precipitation for the entire New England area, the 48-hour value (10.32 inches) being about 1.4 times the next largest obtained at Bartlett, 12 miles south of Pinkham Notch. The increased 48-hour value of 12.90 inches was plotted for one square mile and a straight line drawn through this point and that given for 1000 square miles. Curves for the lesser durations were obtained by drawing lines through the respective 1000-square-mile values and parallel to the 48-hour curve. The resulting depth-area curves were adopted for obtaining maximum rainfall values to apply to the 64 square miles above the Everett Dam and the 426 square miles above the Hopkinton Dam. To these values was added the total of 8.05 inches of snow run-off in 48 hours, proportioned to the rainfall in 6-hour periods. The infiltration rate of 0.05 inch per hour and a base flow of 2 c.f.s. per square mile were adopted. To check the rainfall intensities for one square mile, shown in Plate 10, a mass curve of rainfall was drawn for the March 1936 storm at Pinkham Notch. This is shown on Plate 9 with the corresponding envelope depth-duration curve. The values of the depth-duration curve were increased by 25 per cent and, when compared with those shown on Plate 10, were found to be materially less than the adopted values for the shorter durations.

(c) Controlling Conditions for Spillway Design. Values of maximum rainfall from Plates 8 and 10, together with assumed snow melt values for the spring conditions and the resulting run-off using the adopted infiltration rates of 0.083 and 0.05 inch per hour for summer-fall and winter-spring conditions, respectively, are summarized in the following table:*

*Subsequent to the completion of the above studies, additional maximum precipitation data, as derived for the Ompompanoosuc River in the adjacent Connecticut Basin, were furnished this office. These new data were analyzed and sufficient general agreement found with the above analysis to warrant retaining the selected values.

TABLE 3 - MAXIMUM SUMMER-FALL AND WINTER-SPRING CONDITIONS

Six-Hour Period	Above W.Hopkinton (426 sq.mi.)					Above Everett (64 sq.mi.)				
	Summer-Fall		Winter-Spring			Summer-Fall		Winter-Spring		
	Rain	Run-off *	Rain	Snow Melt	Run-off **	Rain	Run-off *	Rain	Snow Melt	Run-off **
1	0.2	0	0.23	0.22	0.15	0.3	0	0.30	0.25	0.25
2	0.2	0	0.29	0.28	0.27	0.4	0	0.41	0.34	0.45
3	0.6	0.1	0.40	0.39	0.49	0.6	0.1	0.49	0.40	0.59
4	1.1	0.6	0.90	0.87	1.47	1.3	0.8	0.91	0.75	1.36
5	2.0	1.5	1.32	1.28	2.30	2.0	1.5	1.31	1.08	2.09
6	9.4	8.9	3.10	3.00	5.80	12.5	12.0	4.57	3.75	8.02
7	2.2	1.7	1.60	1.54	2.84	2.7	2.2	1.59	1.31	2.60
8	0.8	0.3	0.49	0.47	0.66	1.2	0.7	0.21	0.17	0.08
Total	16.5	13.1	8.33	8.05	13.98	21.0	17.3	9.79	8.05	15.44

Note: All values above are in inches. The sequence of run-off values is arranged, relative to the distribution values for each basin, to obtain the maximum peak discharge.

* Infiltration rate - 0.083 inch per hour

** Infiltration rate - 0.05 inch per hour

The computed spillway floods, based on the foregoing conditions, gave peak reservoir inflows of 110,000 c.f.s. and 104,000 c.f.s. and maximum water-surface elevations of 422.8 and 422.7 for summer-fall and winter-spring conditions, respectively. This difference is somewhat greater for the spillway design flood. The summer-fall conditions were adopted, therefore, as the controlling criteria for spillway design for the Hopkinton-Everett Reservoir.

7. Distribution Values.- (a) West Hopkinton.- The inflow distribution graph for West Hopkinton was initially based on an analysis of the September 1938 flood as it was observed at the power dam located in West Henniker (drainage area 368 square miles). This flood was selected because it is the greatest flood of record on the Contoocook River suitable for determining distribution values. The power dam is approximately 3 miles upstream from the upper end of the reservoir. Thus, the distribution values obtained at this site may be used for deriving inflow hydrographs. Mass curves of precipitation for the 1938 flood are shown on Plate 6. The bi-hourly rainfall values are shown on Plate 7. The total weighted rainfall (Thiessen's Method) was 12.09 inches, while the flood hydrograph, after deducting the base flow, showed a run-off of 6.64 inches.

This indicates that infiltration and other losses amounted to 5.45 inches. By trial and error, the rate of infiltration was computed to be 0.165 inch per hour. Distribution values were then obtained for 6-hour periods for this hydrograph as shown on Plate 11. The distribution graph is plotted on Plate 7. Using these distribution values and the initial rainfall data, the flood hydrograph was reconstructed and satisfactorily checked with the recorded hydrograph. The unit hydrograph, based on one inch of run-off occurring in a period of 6 hours, is shown as a dotted line on Plate 12. Since unit graphs tend to have higher peaks for higher intensities of rainfall, it appeared desirable to increase materially the peak of the unit graph derived from the 1938 flood in order to make it applicable to the much greater run-off values of the computed spillway flood. Hence, the unit graph peak was increased to give a computed spillway flood peak value corresponding to a coefficient "C" of at least 4000 in the relationship $Q = C \sqrt{\text{Drainage Area}}$, as against a value of 2400 for the derived distribution graph. The time of lag was reduced to maintain a reasonable relation to the higher peaked distribution values. The revised unit graph adopted for the computed spillway flood is shown as a solid line on Plate 12.

(b) Everett.- As no hydraulic data were available on the North Branch of the Piscataquog River for the derivation of distribution values, it was necessary to use run-off values from a basin of similar characteristics where discharge records were available. Two basins were studied: (1) the area above the U.S.G.S. gage on the North Branch of the Contoocook River near Antrim, drainage area 55 square miles, and (2) the area above the U.S.G.S. gage on the Smith River near Bristol, drainage area 86 square miles. (The Everett drainage area is 64 square miles.) A study of the drainage basin characteristics indicated that the Smith River was the more comparable to the Piscataquog Basin. Distribution values were then computed for the Smith River gage, based on the 1938 flood, in a manner similar to that outlined for the derivation of the distribution graph for West Hopkinton. The unit hydrograph, based on one inch of run-off occurring in a period of 6 hours, is shown as a dotted line on Plate 13. The drainage area of the Piscataquog Reservoir is less than Smith River, hence the time of lag was decreased approximately 6 hours. The revised unit graph applicable to the computed spillway flood from the Piscataquog Basin is shown as a solid line on Plate 13.

8. Computed Spillway Flood.- The computed spillway flood for Hopkinton-Everett Reservoir was obtained by computing separate spillway floods for the drainage areas above West Hopkinton and Everett Dams and then adding the discharge ordinates of the two hydrographs to obtain the total spillway flood inflow. This is a more severe assumption than taking the values for the total 490 square miles and adjusting for the respective drainage areas. The

computed spillway flood for West Hopkinton was based on the unit graph shown on Plate 12 and the rainfall values shown on Plate 8. As this is a summer storm, an assumed minimum infiltration rate of 0.083 inch per hour was adopted. This is about half that derived by analysis of the 1938 flood and is believed to represent a minimum for the drainage areas considered. A base flow of 2 c.f.s. per square mile was added to the flood run-off. The computations and hydrograph for the computed spillway flood for West Hopkinton Dam are given in Plates 14 and 15. The computed spillway flood for Everett Dam was similarly derived, using the unit graph shown on Plate 13 and the rainfall curves on Plate 8. Rate of infiltration of 0.083 inch per hour and base flow of 2 c.f.s. per square mile were assumed. The computations and hydrograph are given on Plates 16 and 17. The summation of the separate hydrographs representing the total inflow into the reservoir combination is shown on Plate 18. Data pertaining to the computed spillway flood are summarized briefly as follows:

TABLE 4 - COMPUTED SPILLWAY FLOOD

Item	West Hopkinton Dam	Everett Dam	Total Hopkinton-Everett Reservoir
Drainage Area (sq.mi.)	426	64	490
Rainfall (inches)	16.5	21.0	---
Rate of Infiltration (inches per hour)	0.083	0.083	0.083
Run-off (inches)	13.1	17.3	14.1
Volume (acre-feet)*	309,000	60,600	369,600
Peak inflow (c.f.s.)	91,000	23,000	110,000
Coefficient "C" for peak inflow, **	4,420	2,880	4,980
Peak outflow (c.f.s.)	76,000	19,000	95,000
Maximum water surface (ft. above M.S.L.)	422.8	422.7	422.8

* Including base flow

** Coefficient "C" in relation, Peak Discharge = C $\sqrt{\text{Drainage Area}}$

9. <u>Spillway Design Flood</u>.- (a) <u>Reservoir Operation Assumptions</u>.- It is assumed that the reservoir combination will be filled to El. 412, the crest elevation of the spillway in West Hopkinton Dam, at the beginning of the spillway design flood. The pool elevation above Everett Dam will then be 2 feet below its spillway crest. The conduits are assumed to be inoperative.

(b) <u>Spillway Rating Curve</u>.- The spillway rating curves (Plate 19) were computed by the weir formula $Q = CLH^{3/2}$, where values of "C" for an Ogee section were assumed from 3.0 to 3.8, the latter value for the maximum head. Additional head allowance was made by computing losses for the water passages intervening between the spillway control and the pool at which the water surface elevation was desired.

(c) <u>Method of Routing</u>.- As the computed spillway floods for both dams were considered inflows, the flood was routed through the reservoir using the gross storage. A discussion on the method of routing is contained in Appendix B.

(d) <u>Selected Spillway Design Flood</u>.- The total computed spillway flood, with a peak value of 110,000 c.f.s., was routed through the reservoir, obtaining a maximum spillway discharge of 95,000 c.f.s. and a corresponding pool elevation of 422.8. The inflow ordinates of the computed spillway flood were then increased 50 and 100 per cent and similarly routed through the reservoir. The data derived from these three routings are shown graphically on Plate 20, where the per cent of computed spillway flood is plotted, (1) against the pool elevation in feet above mean sea level, and (2) against the peak inflows and outflows. After careful consideration of all factors as discussed in the following paragraph, the computed spillway flood plus a factor of safety of 25 per cent was adopted as the "Spillway Design Flood" for Hopkinton-Everett Reservoir. Inflow, outflow, and stage hydrographs are shown on Plates 21, 22, and 23. Pertinent data are summarized as follows:

<u>TABLE 5 - SELECTED SPILLWAY DESIGN FLOOD</u>
<u>(Computed Spillway Flood plus 25%)</u>

Item	West Hopkinton Dam	Everett Dam	Total Hopkinton-Everett Reservoir
Volume of inflow (acre-feet) (including base flow)	386,000	75,800	461,800
Volume of inflow (inches)	17.0	22.2	17.7
Peak inflow (c.f.s.)	114,000	28,800	137,500
Coefficient "C" for peak inflow	5,540	3,600	6,220
Peak spillway discharge (c.f.s.)	93,000	25,000	118,000
Maximum water surface (ft. above M.S.L.)	424.5*	424.3	424.5*

*Refers to Elm Brook pool elevation (see Plates 20 and 38).

10. Discussion of Factor of Safety.- In selecting 25 per cent as the factor of safety to be applied to the computed spillway flood, the following items, corresponding to those enumerated in paragraph 11 of Engineer Bulletin R. & H. No. 9, 1938, were considered.

(a) The rainfall records and number of storm experiences within the Contoocook and Piscataquog Basins are too few to determine maximum precipitation values. Therefore, the values based on a study of storms over a wide area were adopted (see paragraph 6). The quality and extent of the rainfall records upon which the values were based are such as to require no further allowance for factor of safety for this item.

(b) The adopted maximum precipitation values are greater than maximum recorded values in the northeastern United States. It is believed that there is little possibility that the adopted values will be exceeded, even on small drainage areas. For instance, unofficial records at Peterboro, N.H., and Baldwin, Me., indicate "point" rainfall intensities of 9.5 inches in 2 hours during September 1938 and 11 inches in 3 hours during August 1939. Also, at Tuckerton, N.J., a maximum of 14.81 inches in 14 hours was reported for August 1939. These extreme point rainfall values are lower than the adopted values for one-square-mile drainage area shown on Plate 8.

(c) Data on the maximum summer flood of record (September 1938) were available for determination of the most severe run-off conditions for a summer flood. The average infiltration rate for this flood over the Contoocook Basin was 0.165 inch per hour. As a measure of conservatism, a rate of 0.083 inch per hour (about half the minimum rate experienced) was adopted. Since this rate is equivalent to run-off coefficients of 86 per cent and 89 per cent, respectively, for the maximum 24-hour precipitation above West Hopkinton and Everett Dams, it appears that no further allowance as a factor of safety need be made for this item.

(d) The unit hydrographs were derived from maximum floods. As a further factor of safety, the West Hopkinton graph was peaked, as explained in paragraph 7. Considering the drainage basin characteristics, it is believed that this allowance compensates largely for the inherent errors in applying the unit graphs to the spillway design flood with a peak inflow over 4 times the peak flow of the flood used to derive the distribution values.

(e) Errors in routing the spillway design flood through the reservoir, caused by inaccuracies of assumed storage volume, backwater curves, and time of water travel, are believed to be negligible. Effect of valley storage in the reservoir is eliminated, since the computed floods represent inflow hydrographs. Because the reservoir consists of two storage areas connected by a canal, errors are introduced in the spillway routing computations through use of an average pool elevation in the Hopkinton section and the Everett section, instead of the individual pool elevations for the given discharge condition. These differences in elevation have been corrected by checking back through the routing and computing the difference in head on either side of the canal under the severest conditions of flow that occur. It is believed that only a small allowance is required as a factor of safety for this item.

(f) The criterion was adopted that the outlet conduits would be inoperative and the reservoir full at the beginning of the flood. Power development at the reservoir in the future would not introduce a more severe condition. No allowance need be made for this item.

(g) The spillways at both West Hopkinton and Everett Dams are straight overflow Ogee sections. The spillway discharge capacities have been computed with conservative discharge coefficients, and it is believed that the actual spillway capacities will exceed the computed. No further allowance need be made for this item.

(h) The freeboard storage in Hopkinton-Everett Reservoir amounts to about 2 inches over the drainage area controlled. This, with the long spillways provided, affords a large factor of safety against overtopping, since a flood in excess of 200 per cent of the computed spillway flood would be required to reach the top of the dams.

11. Comparison with Maximum Flows of Record.- The maximum peak flow of record at West Henniker (equivalent to inflow to the West Hopkinton Dam) was 29,000 c.f.s. in March 1936. The estimated maximum peak flow at the Everett dam site for the same flood is 6,000 c.f.s. The peak inflows of the adopted spillway design floods for these dams are, respectively, 3.9 and 4.8 times the maximum known flows. The peak inflows of the adopted spillway design floods for West Hopkinton and Everett Dams, together with the values previously adopted for the Franklin Falls and Blackwater Reservoirs in this district, are plotted on Plate 24 for comparison with the maximum recorded discharges for the entire New England area.

12. Freeboard.- The amount of freeboard required as allowance for wave action, ride-up of waves, and wind set-up has been computed in accordance with Engineer Bulletin R. & H. No. 9, 1938. The results of the computations, together with the amount of freeboard actually provided under the proposed design, are summarized in the following table:

TABLE 6 - FREEBOARD REQUIREMENTS

1	2	3	4	5	6	7	8	9
Structure	Max. Fetch "F"	Max. Velocity "V"	Wave Height "h"*	Wave Plus Ride-up 1.4h	Depth of Water "D"	Wind Set-up "S" **	Amount of Freeboard Required by formula 1.4h + S	Actual Freeboard Provided
	(mi.)	(mph)	(ft.)	(ft.)	(ft.)	(ft.)	(ft.)	(ft.)
W.Hopkinton Dam	2.5	80	3.6	5.0	40	0.5	5.5	5.8
Dike H-2	2.5	80	3.6	5.0	40	0.5	5.5	5.5
Dike H-3	3.0	80	3.9	5.4	40	0.6	6.0	6.2
Everett Dam	3.9	80	4.1	5.7	100	0.3	6.0	5.7
Dike P1	1.5	80	3.3	4.6	35	0.3	4.9	5.7
Dike P2	3.9	80	4.1	5.7	30	1.0	6.7	6.7

* by formula, $h = 0.17\sqrt{VF} + 2.5 - \sqrt[4]{F}$

** by formula, $S = .00125 \frac{V^2 F}{D} \cos A$ (A assumed to be zero)

13. Determination of Reservoir Capacity.- (a) Reservoir Design Flood.- The following flood, similar in magnitude and manner of occurrence to that of March 1936, was used as the basis for the initial determination of storage and discharge capacities for all reservoirs in the Merrimack Basin.

Order of Days	Rainfall in Inches	Run-off (90%)
1	1.5	1.35
2	1.5	1.35
3	1.5	1.35
4	0	0
5	0	0
6	0	0
7	1.17	1.053
8	6.00	5.40
9	1.17	1.053
Total	12.34	11.556

Since the primary purpose of the design flood is to determine the storage and corresponding discharge capacities necessary to control the given volume of run-off, it was considered satisfactory to use composite distribution values, typical of the more flashy tributaries in the Merrimack Basin. The relation between the total reservoir outlet discharge capacity and the storage necessary to control the design flood are shown on Plate 25. Also shown is the adopted distribution of the required total maximum outlet discharge between West Hopkinton and Everett Dams. This distribution is governed by the maximum discharge that can be passed down the Piscataquog River from Everett during flood control operation.

(b) Economic Capacity.- As the second step in selection of capacity, the flood control benefits were determined for various amounts of storage in Hopkinton-Everett Reservoir. For this step, the reservoir was considered in combination with Franklin Falls and Blackwater Reservoirs and total annual benefits of the system determined. The total annual benefits, corresponding annual costs, and resulting ratios of benefits to costs are plotted against the various storage capacities for Hopkinton-Everett Reservoir on Plate 26. The curve of ratios of benefits to costs on Plate 26 shows that the optimum capacity of the reservoir from an economic standpoint lies in the range from 5.7 to 6.1 inches, with an optimum point at 6.0 inches.

(c) Selected Storage and Discharge Capacities.- Based on the foregoing economic analysis, plus consideration of the desirability of a high degree of control, a capacity of 157,000 acre-feet (6.0 inches), obtainable with a pool elevation of 412, was adopted. From Plate 25, the corresponding total design discharge (at full pool elevation) is 8,700 c.f.s. taking the storage requirement of 6 inches over the total 490-square-mile drainage area and 7,500 c.f.s. when giving reasonable consideration to the effect of Mountain Brook and West Peterboro Reservoirs. Fixing the maximum design outlet discharge for the Everett Dam at 3,000 c.f.s. leaves between 5,700 and 4,500 c.f.s. to be discharged from the West Hopkinton Dam. A compromise value of 5,000 c.f.s. was taken as the maximum outlet design discharge for the West Hopkinton Dam. Because of the dual nature of the reservoir, it is possible to reduce flood stages downstream to a greater degree by regulation of the discharge than through uncontrolled operation. This method of operation is discussed in detail in Appendix B, and its effects are illustrated with the test floods described in the following paragraph. The effect of such operation is to minimize the importance of design outlet capacity as long as sufficient excess capacity is provided for emptying.

(d) Test of Adequacy of Reservoir Capacity.- Two severe floods were used to check the adequacy of the reservoir design: (1) a highly peaked flood representing severe summer and fall conditions, and (2) a large-volume winter-spring flood including snow run-off. The summer flood was based on the September 1932 rainfall, the greatest of record in New England (see Plate 27). For a winter-spring flood, the March 1936 flood, the greatest of record, was selected. With unregulated design discharge, the reservoir will control both of these floods without spillway discharge. Under the proposed plan of regulated discharge, which gives greater benefits downstream (see Appendix B), the reservoir is subjected to a more severe test of adequacy. The effect of the reservoir on the two maximum floods, with regulation as proposed, is illustrated on Plates 28, 29A, and 29B. In both cases there is a small spillway discharge at the end of the flood period, but the net control of the floods is more complete because a greater reduction has been afforded to that part of the hydrograph which adds to the peak on the lower Contoocook and on the Merrimack River. The spillway discharge is obtained at a time when it is not synchronized with the flood peaks downstream.

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX B

HYDRAULICS

Table of Contents

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX B

HYDRAULICS

1. Factors Involved.- The hydraulic design of Hopkinton-Everett Reservoir presented six principal problems: (1) distribution of the spillway discharge between the two dams, (2) distribution of the outlet discharge between the two dams, (3) determination of the discharge capacity of the required connecting canals, (4) selection of method of operation of the reservoir for maximum utilization of its capacity, (5) design of the system for utilizing the Hopkinton section to control as much as possible the floods from the Contoocook, requiring a minimum of diversion of Contoocook River flows down the Piscataquog River, and (6) the evolvement of a satisfactory method of routing the floods through the reservoir, taking into account two inflow hydrographs, two outflows (outlet and spillway discharges), two reservoir capacities, and the discharge from one section of the reservoir to the other through the connecting canals.

2. Spillway Requirements.- (a) General Plan of Spillways.- Since there are two natural outlets for the reservoir, the Contoocook and Piscataquog Rivers, there are three possible distributions of the total spillway discharge: (1) a spillway at West Hopkinton Dam and none at Everett, (2) a spillway at Everett Dam and none at West Hopkinton, and (3) spillways at both dams. The first alternative would be possible without appreciable increase in size of Canal No. 2 but would require a 25 per cent increase in thelength of the Hopkinton spillway. The second alternative is impracticable because of the great increase in size required for Canal No. 2. The third alternative, with spillways at both dams, was adopted. This provides protection to both dams in the event that the connecting canal (No. 2) becomes partially ineffective for discharge, and, for floods approaching in magnitude the spillway design flood, eliminates the objectionable feature of diverting the total flood down one of the river channels with discharges in excess of those contributed by the basin into which the diversion is made.

(b) Criteria for Distribution of Discharge between Spillways.- Determination of the respective discharge capacities of the spillways was based on two considerations: (1) For the spillway design flood, the maximum discharge from each spillway should be approximately in proportion to the peak inflow from the drainage area tributary to its respective dam; (2) In operating the reservoir as described later in paragraph 6(b) for all floods up to a magnitude comparable to the maximum recorded (March 1936), no spillway discharge should occur at the Everett Dam, or, in the event of some spillway discharge, the Everett outlets should be closed to keep the total outflow down the North Branch Piscataquog to about 3,000 c.f.s. until the flood is over and conditions downstream will permit the safe passage of the increased outlet discharge needed to expedite emptying the reservoir.

(c) Selected Distribution of Spillway Capacity.- In order to meet both conditions outlined above within limits of the desirable size of spillways from a structural and economic standpoint, it was necessary to set the elevation of the Everett spillway (El. 414.0) 2 feet above West Hopkinton spillway (El. 412). With respect to the first criterion above, the selected spillway dimensions give the following ratios of outflow to inflow for the maximum condition (the spillway design flood):

	West Hopkinton	Everett
Peak inflow	114,000 c.f.s.	28,800 c.f.s.
Peak spillway outflow	93,000 "	25,000 "
Ratio of peak outflow to inflow	0.82	0.87

With respect to the second consideration, theproposed distribution of spillway capacities was checked by routing both the 1936 flood and that based on the 1932 storm through the reservoir using the proposed reservoir operation. In neither case was the total outflow from Everett in excess of 3,000 c.f.s. (See Plates 28, 29A and 29B.) The respective spillway discharges in c.f.s. per square mile for various reservoir stages are shown on Plate 30. Both spillways have equal unit discharges (53 c.f.s. per square mile) for a total discharge of 26,000 c.f.s., which is in the range covered by condition (2) above. For lesser floods, the West Hopkinton unit discharge exceeds that of Everett, thus preventing diversion of the preponderant Contoocook River floods down the Piscataquog. The Everett spillway is required only for those rare floods ranging from the maximum of record to the predicted maximum possible.

(d) Physical and Structural Considerations.- Consideration of cost of the dams and conditions upstream showed that the top of the dams should be at about El. 430. Study of spillway capacities required to pass the spillway design flood with adequate freeboard allowance and the physical characteristics at both sites indicated a spillway at West Hopkinton 650 feet long, effective length about 600 feet, crest at El. 412, and maximum surcharge of 11.8 feet, and at Everett, a spillway with length of 200 feet, crest at El. 414, and maximum surcharge of 10.3 feet. The natural features at the site of the West Hopkinton spillway permit a long, free overfall spillway on rock with adequate approach channel, but restrict the spillway discharge channel to a width of about 300 feet and the depth of flow to one that could be kept within the available rock cut or a reasonable height of channel wall. The spillway will make an angle of 30 degrees with the direction of the spillway discharge channel and will be set back at its downstream end to maintain the full 300-foot effective width of channel. A secondary weir will be placed in the spillway discharge channel of such height that, for all flows, sufficient depth is provided to produce a hydraulic jump below the spillway, but not too high to submerge the spillway. This plan will cause redistribution of flow from the spillway, giving a reasonably uniform flow over the secondary weir and across the spillway discharge channel. Without such a plan, the unrestrained discharge over the spillway would impinge on the left channel wall and encroach on the right portion of the spillway discharge channel, requiring much greater channel depths to take care of excessive wave action and disturbance to flow. These conditions would also affect the design of the stilling basin. The Everett spillway will be entirely on rock and of conventional design, having an adequate discharge channel in rock with no dangerous erosion problem involved in returning the flow to the river channel. Both spillways are designed for free overfall with crest profile shaped to prevent negative pressures under maximum surcharge. The basic rating curves were computed from the weir formula with "C" values varying from 3.0 to a maximum of 3.8 at full head (see Plate 19).

3. Outlet Discharge Requirements.- (a) General Plan of Outlets.- The dual reservoir, controlling two tributaries, both affording natural outlets, makes possible several plans of discharge distribution and operation. It is possible either to pass all the outlet discharge down only one of the rivers, or to distribute the discharge between the two. The latter alternative was adopted because of the limited discharge capacity of the Piscataquog River and because greater benefits could be realized from such a separation of discharges.

(b) Distribution of Discharge between Dams.- Capacity studies, described in paragraph 13 of Appendix A and as illustrated on Plate 25, indicate that, for a storage of about 6 inches, the total reservoir discharge should be about 8,000 c.f.s. The geographic location of the two rivers and the time of arrival of their flood peaks at the Merrimack River are such that whatever flow is bypassed down the Piscataquog River causes corresponding reductions at the damage centers on the Contoocook River, and at Concord and upper Manchester on the Merrimack River. For maximum flood control, therefore, it is desirable to discharge as much as possible down the Piscataquog River without increasing flood conditions in the lower reaches. It is estimated that the maximum flow of record on the North Branch Piscataquog was at least 90 c.f.s. per square mile. A maximum outlet design discharge of 3,000 c.f.s. (47 c.f.s. per square mile) was selected for Everett Dam. Thus, flood conditions on the Piscataquog will be improved during flood control operation of Hopkinton-Everett Reservoir. In addition, the outlet capacity of Everett Dam may be doubled (to 6,000 c.f.s.) for emptying purposes without causing worse flooding on the North Branch than would have been experienced if the flood were uncontrolled. Farther downstream, on the main Piscataquog, the emptying discharge of 6,000 c.f.s. will not make flood conditions worse because the flow of the North Branch will have been desynchronized with the remainder of the Piscataquog. Adopting the design discharge of 3,000 c.f.s. for Everett Dam leaves about 5,000 c.f.s. for West Hopkinton Dam in order to control the reservoir design flood with a storage capacity of 6 inches.

(c) Time Required to Empty Reservoir.- Sufficient outlet capacity will be provided in both dams to double theoutlet design discharge for emptying purposes. With a full pool at El. 412, the emptying capacity of the West Hopkinton outlets will be 10,000 c.f.s., and that of the Everett outlets, 6,000 c.f.s. The proposed emptying schedule is shown for the 1936 flood on Plate 29B. The volume in the reservoir above El. 400 (crest of canal weir) can be drawn down by the discharge from either dam or both, but below El. 400 it is necessary to discharge the remaining storage from the separate sections of the reservoir. At El. 400 this division of storage is 28,000 acre-feet in the West Hopkinton section of the reservoir and 65,000 acre-feet in the Everett section. As shown on Plate 29B, 6 days are required to empty the West Hopkinton section from full pool (El. 412) and 12 days to empty the Everett section. The reservoir will be drawn down to El. 400 in 2-1/2 days. The effect of the canal weir in lengthening the time of emptying the Everett section by cutting off the excess Hopkinton outlet capacity below El. 400 was found to be small. No appreciable overall saving in time of emptying was shown by computations assuming no canal weir, due to the effect of head losses on canals Nos. 1 and 2, and the fact that the net outlet discharge of both sections below El. 400 tends to bring the water surface in the two sections down near enough together to leave little head for inducing flow from Everett to Hopkinton. Plate 39 shows the storage available during the emptying period. Three inches, or one-half of the total storage, are recovered in 3 days. Five inches, or 83% of the total, are recovered in less than 7 days.

(d) Physical and Structural Considerations.- (1) Two gated conduits will be provided for the West Hopkinton section of the reservoir and will be located in the dam on the Contoocook

River. The conduits are designed for 5,000 c.f.s. capacity each when discharging with full reservoir (El. 412). Elevation of the conduit invert was based on consideration of operation for the downstream Davis Mill pond so as not to interfere with draw-down demands. This consideration required lowering the invert below that desirable for best outflow conditions, placing the conduits at El. 370, or 7 feet below the crest of the Davis Mill Dam. This will require further study to give a safe stilling basin design. Each conduit will have a separate stilling basin. The conduit cross-section will be horseshoe-shaped, with an area of 130 square feet. Two gates, 7'x10', will be provided in each conduit. Outlet and tailwater rating curves are shown on Plate 31. The outlet curve was computed assuming a coefficient "n" = 0.013 for the frictional losses in the conduit. The tailwater curve was based on computations for the discharge capacity of the Davis Mill Dam.

(2) The outlets for the Everett Dam will consist of two conduits, located on bedrock, through the earth section of the dam. A single gate in each conduit will permit control of the outlet discharge in accordance with the proposed plan of reservoir operation. A dual stilling basin will be provided to dissipate the high velocity of outflow from the conduits since the cost will be no more than that required to lead the uncontrolled outflow a safe distance away from the dam. Outlet and tailwater rating curves are shown on Plates 32 and 33.

4. <u>Design of Canal No. 1</u>.- (a) <u>Limiting Conditions</u>.- (1) Locating the West Hopkinton spillway near the Elm Brook pool places the limiting condition on the design of Canal No. 1 that it be able to pass safely the peak of the spillway design flood of about 100,000 c.f.s. from the Contoocook River pool.

(2) The canal capacity must be sufficient to pass the reservoir design discharge of about 28,000 c.f.s. with Elm Brook pool at spillway crest without excessive velocity or head losses.

(3) The canal invert must be low enough to provide for emptying the Elm Brook pool through the West Hopkinton outlets.

(b) <u>The first condition</u> above, together with the physical criterion for canal side slopes of 1 on 2-1/2, led to the adoption of a minimum canal bottom width of 120 feet at the Contoocook River end and 150 feet near the Elm Brook pool end. The direction of flow for maximum discharge permitted reduction of canal width in the upstream direction, maintaining practically constant mean velocities, since the backwater rise gives additional depth to compensate for reduction in width. The maximum mean velocity is less than 10 feet per second. It will be cheaper to protect against this velocity by riprapping and providing the required additional height of West Hopkinton Dam for the greater velocity head and losses than it will be to excavate to a size that would prevent non-eroding velocity in an unlined canal. The water surface profile and mean velocities along Canal No. 1 for a discharge of 100,000 c.f.s. are shown on Plate 35.

(c) The second condition above is amply provided for by the requirements of the first condition. The water surface profile and mean velocities along Canal No. 1 for a discharge of 28,000 c.f.s. are shown on Plate 35.

(d) The third condition above required that the canal invert be no higher than El. 380.

(e) The rating curves for Canal No. 1 are shown on Plate 34 for conditions (1) and (2) above. During the spillway design flood the discharge is essentially from the Contoocook River only, and the Hopkinton spillway becomes the primary control for Canal No. 1. The backwater as affecting the stage in the pools intervening between the spillway and Contoocook River is computed on the assumption that the inflow of the Contoocook River equals the outflow over the spillway. This is not entirely correct, due to the effect of the intervening storage and also some possible diversion into Everett through Canal No. 2. The pool elevations shown on Plate 34, however, are the maximum for any given discharge. For maximum discharge values corresponding to the reservoir design flood (or for other floods of lesser magnitude that do not overtop the spillways), the primary control for Canal No. 1 is the weir in Canal No. 2. The maximum discharge in Canal No. 1 is reached when the inflow from the Contoocook River equals the outflow from Canal No. 2. For the reservoir design or lesser floods this maximum discharge is passed before the discharge of the weir in Canal No. 2 is affected by high pool elevations in the Everett section. Reservoir conditions during any given flood must first be evaluated by reservoir routing (see methods A and B, par. 6). The rating curves given on Plate 34 (as well as those for Canal No. 2 on Plate 36) serve as a check on flow conditions assumed during the cut-and-try method of routing. On obtaining any maximum discharge value, the rating curves give the corresponding pool elevation on which the design of structures must be based.

5. Design of Canal No. 2.- (a) General Considerations.- The effective utilization in the Hopkinton-Everett Reservoir of the greater storage available in the Everett section is largely dependent on having adequate discharge capacity for the connecting canal (No. 2). The selection of the canal invert was dependent primarily on physical conditions affecting its cost. On the basis of seismic exploration, a satisfactory adjustment of cost and required capacity was obtained with the canal invert at El. 385. An approach to the optimum canal capacity was made by considering the canal operation, (1) for a flood comparable to the reservoir design flood, and (2) for a flood comparable to the spillway design flood. During normal flood control operation, particularly if the reservoir is to be operated for maximum downstream benefits, the canal must be able to pass flows increasing nearly up to the maximum Contoocook River inflow, without causing the pool elevation to exceed the West Hopkinton spillway crest. For the spillway design flood, where both spillways have capacities adjusted to take care of the flood from their respective drainage areas, the canal will

not be required to pass excessive flows from one reservoir section to the other. Also, for a spillway design flood, assuming both reservoirs at El. 412 at the beginning of the flood, there is immediately available a large canal capacity. The above considerations indicated that the reservoir design flood conditions would govern the capacity required; hence the canal capacity was approximated on the basis of passing about 28,000 c.f.s. with the West Hopkinton pool at El. 412.

(b) Another consideration affecting the design of Canal No. 2 is the desirability of controlling Contoocook River floods as much as possible using only the limited storage available in the West Hopkinton section of the reservoir. A satisfactory reduction of all floods up to a frequency of from 15 to 20 years can be obtained on this basis by placing a weir near the downstream end of the canal with crest at El. 400, at which elevation there are 28,000 acre-feet of flood control storage in the West Hopkinton section. The weir in Canal No. 2 is also desirable in order to limit maximum canal velocities, thereby minimizing canal maintenance.

(c) Physical and Structural Considerations.- The proposed canal will have no slope longitudinally, for flow will occur in both directions and the use of the weir offsets any hydraulic advantage that might be gained by an invert slope from West Hopkinton to Everett. The canal invert was placed at El. 385, which will minimize the rock excavation both on the bottom and sides and yet give a cross-sectional shape that is good hydraulically. The minimum cross-sectional area, based on cost and required capacity, was determined to be 5,625 square feet at El. 412. A practically constant section is possible throughout its length up to El. 420 and the alignment will be made on long curves where required. Some overflow area will be obtained above El. 420 where the natural valley widens out, but losses due to such changes in section and curvature in alignment were considered small and compensated for in the roughness coefficient assumed in the backwater computations.

(d) The discharge capacity of Canal No. 2 was computed by combining the rating curve of the control weir with backwater computations from the weir to the Elm Brook pool. The rating curve of the control weir (Plate 36) was computed by the weir formula $Q = CLH^{3/2}$, with values of "C" varying from 3.0 to 3.8 for the maximum head. Backwater computations were then made, allowing for frictional losses and changes in velocity head, to obtain the pool elevation at the upstream end. For pool elevations exceeding critical depth at the downstream end of the canal, similar backwater computations were made, starting with this assumed pool elevation, and obtaining the corresponding discharge and upstream pool elevation. For computing canal losses, a value of Kutter's "n" of 0.035 was used. The rating curves for the canal, showing the pool elevations at the upstream and downstream ends of the canal for various conditions of discharge, are given on Plate 36. These curves illustrate flow conditions in the direction from West Hopkinton to Everett, since this is both the most frequent as well as governing direction of flow. A backwater curve and corresponding mean velocities in the canal for a discharge of 28,000 c.f.s. are given on Plate 37.

6. Reservoir Operation.- (a) Relation to Comprehensive Plan.- The operation of Franklin Falls, Blackwater, and Hopkinton-Everett Reservoirs will result in major control over the two largest tributaries of the Merrimack Basin and will change the flood situation in the basin to the extent that some consideration must be given to the remaining uncontrolled tributaries in determining possible operation of the Hopkinton-Everett Reservoir. With control of the Pemigewasset, Blackwater, the Contoocook above West Hopkinton, and part of the Piscataquog, the uncontrolled flood producing tributaries above Nashua, N. H., would be reduced to: (1) Winnipesaukee River, (2) Warner River, (3) Souccok River, (4) Suncook River, (5) South and Middle Branch of the Piscataquog River, and (6) Souhegan River. Past floods of record, particularly the 1936 and 1938 floods, have demonstrated that, due to the pattern of the basin, the flood peak flows from these tributaries reach the Merrimack River and pass downstream before the principal flood peak, originating on the Pemigewasset and Contoocook Rivers, arrives. Hence, with reservoirs controlling the two largest tributaries, the "center of gravity" of the flood producing basin would move a considerable distance downstream. It now may be conceived that if the 1936 or 1938 flood were reproduced with reservoir control, the initial peak flow at Nashua, for example, would be caused by discharges from the Souhegan, Piscataquog, and Suncook Rivers, with the flow from the Winnipesaukee, Warner, and Soucook Rivers, and the reduced discharges from the reservoirs arriving on the falling side of the initial peak and causing a secondary peak. The secondary peak might be greater than the first for extreme floods having a large spillway discharge at Franklin Falls. Hence, the time between the storm occurrence and the peak discharge remaining that is produced by the lower tributaries will be much less with reservoirs than without reservoirs, and, conversely, the time of lag of the peak flow from the controlled tributaries will be appreciably greater.

(b) Proposed Plan of Operation.- In view of the foregoing drainage basin characteristics, the geographic location of Hopkinton-Everett Reservoir is favorable to a plan of discharge regulation. Briefly, the plan is to hold back all discharges from the reservoir during the beginning of the flood and to allow the flood in the lower portion of the basin to form and to pass down the river. Plates 28 and 29A illustrate how this would be accomplished during the hypothetical flood based on the 1932 storm and the recorded 1936 flood. It is evident th t if the outlets are discharging during the peak of the flood, that from West Hopkinton Dam would add to the natural peak flow from the Warner River, and discharge from the Everett Dam would have a similar effect on the Piscataquog River. Temporarily closing off the total reservoir discharge of 8000 c.f.s. will reduce the uncontrolled flow on both the Contoocook and the Piscataquog Rivers and add materially to the downstream benefits. It is believed that this is a justifiable regulation for this reservoir, due to its capacity of 6.0 inches and its location in the Merrimack Basin. Tested with the single high peak flood based on the 1932 storm and the double peak of the 1936 flood, the regulation is found to give satisfactory results. Holding back floods of this magnitude resulted in some spillway discharge, but coming at the end of the flood it causes no appreciable damage immediately downstream on the Contoocook, giving a lower peak value than that for the uncontrolled drainage area of the Contoocook. It was also found that, due to the extended run-off from the Contoocook Basin, the spillway discharge for the 1936 flood came even later than the corresponding spillway discharge from Franklin Falls Dam. In view of the pattern and characteristics of the Merrimack Basin, it is expected that this non-synchronization of peaks will occur during all floods which the reservoir system is designed to control.

7. Method of Reservoir Routing.- (a) General.- The basic principles of reservoir routing as applied to a single reservoir are used in the dual reservoir formed by the West Hopkinton and Everett Dams. However, instead of operating with but one set of inflow, outflow and reservoir storage values, the dual reservoir routing is much more complex in having to deal with two sets of these values considering also the relation to the values of the discharge in the connecting canal. Two methods were used in routing floods through the dual reservoir, method "A" being utilized for outlet studies with the reservoir empty at the beginning of the flood, and "B" a method satisfactory for spillway studies where the reservoir is assumed to be full at the beginning of the flood. The following basic formulae for reservoir routing apply to both methods:

$$I = O \pm \Delta S \qquad (1)$$

where: I = inflow in acre-feet

O = outflow in acre-feet

ΔS = increment of storage in acre-feet

At either the West Hopkinton or Everett sections outflow consists of the outlet or spillway discharge plus or minus the discharge through the canal.

$$\text{Hence: } O = D \pm C \qquad (2)$$

where: D = outlet or spillway discharge in acre-feet

C = canal flow in acre-feet (Canal No. 2)

Then the basic formulae for these two reservoirs in combination are:

$$I_h = D_h \pm \Delta S_h \pm C_h \qquad (3)$$

$$I_e = D_e \pm \Delta S_e \mp C_e \qquad (4)$$

where the subscript "h" refers to West Hopkinton and "e" refers to Everett.

(b) Method "A".- Formulae 3 and 4 are used in method "A" for routing of the 1936 flood (Plates 29A and 29B) and the flood based on the storm of 1932 (Plate 28). Because of the number of variables involved it is necessary to use short increments of time in order to obtain and balance the values for each term. This is most important for the canal flow which varies considerably for small changes in reservoir elevations, and short time increments are necessary to ascertain the respective reservoir elevations with sufficient accuracy. Time increments of one-tenth of a day were found to give satisfactory results. Each term in the two formulae is a variable, and, except for the inflows, all can be expressed graphically as a function of the water-surface elevations in the respective reservoirs. The hydrographs, storage curves, and discharge rating curves used to solve for each variable in routing the 1936 flood are as follows:

I_h = flood inflow to West Hopkinton	-	Plate	29A and B
D_h = outlet discharge from West Hopkinton	-	"	31
& spillway discharge from West Hopkinton	-	"	19
ΔS_h = West Hopkinton storage increment	-	"	3
$C_h = C_e$ = Canal flow (Canal No. 2)	-	"	36
I_e = flood inflow to Everett	-	"	29A and B
D_e = outlet discharge from Everett	-	"	32
ΔS_e = Everett Reservoir storage increments	-	"	3

The actual computations consist of satisfying the conditions in formulae 3 and 4 for each time period and are most readily accomplished by means of an extensive tabulation.

(c) Method "B" is used in routing a flood when both sections of the reservoir are practically full at beginning of the flood. Method "B" differs from "A" in that the two sections of the reservoir act as a single reservoir assuming that the water-surface elevation is the same in both during the entire duration of the flood. Since, at full pool, large canal flows occur with only small differences in reservoir levels, this assumption does not appreciably affect the value for maximum reservoir elevation and can be checked to the required accuracy as explained below. The basic formulae used in method "A" are also applicable to method "B". The formulae are as follows:

$$I_h = D_h \pm \triangle S_h \pm C_h \tag{3}$$

$$I_e = D_e \pm \triangle S_e \mp C_e \tag{4}$$

In order to simplify algebraic signs, consider the first part of the flood in which both sections of the reservoir are storing and canal discharge is from West Hopkinton to Everett.

Then $$I_h = D_h + \triangle S_h + C_h \tag{5}$$

$$I_e = D_e + \triangle S_e - C_e$$

or $$I_e + C_e = D_e + \triangle S_e \tag{6}$$

$$C_h = C_e$$

$$C_e = D_e + \triangle S_e - I_e \tag{7}$$

Substituting in (5)

$$I_h = D_h + \triangle S_h + D_e + \triangle S_e - I_e \tag{8}$$

$$(I_h + I_e) = (D_h + D_e) + (\triangle S_h + \triangle S_e) \tag{9}$$

or $$\Sigma I = \Sigma D + \Sigma \triangle S \tag{10}$$

Equation (10) is the basic equation for reservoir routing, combining equations (3) and (4). The routing computations, therefore, consider the two sections of the reservoir as a single unit, and the inflows, discharges, and storage curves are combined rather than used individually. It is possible to obtain this simplification, because the reservoir outflows, D, and the storages $\triangle S$, are assumed to be functions of the same elevation and the small difference in reservoir levels required to give the canal discharge is made up in the time interval chosen for the routing. Routing proceeds in the usual method for a single reservoir. The graphs necessary for this routing, as applied to the spillway design flood, are as follows:

I	=	total spillway design flood inflow	- Plate 23
D	=	total spillway discharge	- Plate 19
S	=	storage increments obtained from total curve	- Plate 3

It is now possible to determine the canal flows and correct for the assumption that the two sections of the reservoir are at the same level. A graph of reservoir stage vs. time is first drawn from the combined routing computations. The canal flow, C, is then computed by means of equation (3) or (4), (the former, applying to the West Hopkinton section, is used in the following explanation). For each time period, (1) the value of I_h from the inflow hydrograph, (2) the approximate value of D_h from the spillway rating curve for the given reservoir stage, and (3) the value of $\pm \Delta S_h$ from the storage curve and the given reservoir stage may be obtained. The difference in volume between the inflow and the combined outflow and storage must be the volume passing through the canal. Similarly, canal volumes in acre-feet may be obtained for every period of the routing. Either equation (3) for West Hopkinton or equation (4) for Everett may be used in these computations. Converting the volumes to discharge rates and plotting as bar values, the canal discharge hydrograph can be drawn. The next step is to ascertain the difference in pool elevation required for the given canal flow. For example, the maximum water surface attained from routing the spillway design flood by method B was elevation 424.5 at 3 P.M. on the third day. The canal flow at this time, as derived from equation (3), was 10,000 c.f.s. from Hopkinton to Everett. This indicates that, for this discharge and pool elevation 424.5 at West Hopkinton, the difference in pool elevation is 0.2 foot. As the reservoir elevation of 424.5, obtained from the combined routing, is primarily due to the discharge from the West Hopkinton spillway, the computed elevation is considered as applying to the Elm Brook pool. The Everett pool will then be 0.2 foot lower, at El. 424.3. This small difference in elevation means only a difference of 700 c.f.s. in the Everett spillway discharge and is considered close enough to justify use of method B. (The maximum canal flow for the spillway design flood is 18,000 c.f.s. and requires only 0.3 foot difference in the elevation of the pools.) To determine the water surface elevations in the Contoocook River pool and the spillway pool, it is necessary to use Plate 34, applying the values computed at Elm Brook pool. The inflow from the Contoocook River at the time of the maximum water surface elevation was about 92,500 c.f.s. (Plate 21). As inflow practically equals outflow at the maximum stage, it is assumed that the flow through West Hopkinton is constant. From Plate 34, for a constant discharge of 92,500 c.f.s., the corresponding pool elevations are:

Contoocook River pool	-	426.2
Elm Brook pool	-	424.5
Spillway pool	-	423.8

A higher rate of discharge will occur through Canal No. 2 before maximum water surface elevation is reached, probably occurring about the time of the inflow peak. However, except for producing slightly higher velocities in the canal, this maximum canal discharge will have no appreciable effect on any design criteria. The foregoing general explanation is also applicable to method A wherein it is apparent that small differences in water surface elevations between Contoocook River pool and Elm Brook pool are neglected. To determine this difference for each step of a routing would constitute an unnecessary refinement. It is essential, when using equations (3) or (4), to use the proper algebraic sign for both the storage increment and the canal discharge volume. In either equation, a plus C signifies a canal outflow, and a minus C denotes a canal inflow.

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

SUPPLEMENT TO APPENDIX B - HYDRAULICS

March 4, 1941

DISCUSSION OF CAPACITY OF CANAL NO. 2

8. Reference is made to a memorandum to Mr. McAlpine from Mr. G. A. Hathaway, dated October 15, 1940, attached to the 2nd indorsement of the letter submitting the definite project report for Hopkinton-Everett Reservoir (E.D. File No. 7402(Merrimack R., Hopkinton-Everett Res.)-37). In this memorandum it was recommended "that further consideration be given to increasing the capacity of the canal between the two reservoirs so that a more flexible flood control operation may be assured."

9. Original Basis for Selection of Capacity.- The following criteria entered into the selection of the capacity of the canal:

(a) Complete utilization of all storage in both sections of the reservoir for the reservoir design flood.

(b) Criterion (a) above to be accomplished without spillway discharge for regular retarding basin operation and, for gate operation as proposed, to have spillway discharge at Hopkinton no greater than downstream channel capacity.

(c) To eliminate or limit diversion of ordinary (15 to 20-year frequency) Contoocook River floods into the Everett section of the reservoir.

(d) To obtain the required cross-sectional area of canal with a minimum of rock excavation.

10. Selected Canal Capacity.- The inflows, outflows, and canal discharges, without gate regulation, for the reservoir design flood are shown on Plate 61. Similar data showing the effect of gate regulation are shown on Plate 62. In accordance with criteria (a) and (b), there is no spillway discharge without gate regulation, and the spillway discharge at Hopkinton with regulation is only 10,000 c.f.s., which does not exceed channel capacity and which occurs on the receding side of the flood in such a manner as not to add to the main stream flood peak at the principal damage centers.

11. Effect of Selected Reservoir Design on Flood Based on September 1932 Storm.- In order to check the selected capacities for outlets and canal, a flood based on the September 1932 storm was routed through the reservoir. This flood (see Plate 28) is about 25% greater than the reservoir design flood and the maximum floods of record. The reservoir operation during this flood is questioned in the memorandum referred to in paragraph 8. It is of interest to note that during a large single peak flood, or during the second peak of a flood similar to that of March 1936, there is a tendency to form two peaks in the Hopkinton pool stage graph, shortly after occurrence of the maximum inflow peak. Only during the first peak is the maximum canal capacity available. During the second peak, filling of the Everett Reservoir submerges the canal and rapidly reduces its discharge capacity. Hence, there must always be storage available in Everett during the first rise in the Hopkinton pool. Where this first rise causes appreciable overtopping of the spillway, there is a valid objection in principle to the limited canal capacity causing it, but this does not mean that the total reservoir capacity is not utilized in the control of the flood. Any flood greater than the reservoir design flood will exceed in volume that provided in the reservoir and produce a spillway discharge, regardless of the reservoir considered. The unique characteristic of the Hopkinton-Everett combination should not be unduly penalized with excessive canal cost where this initial spillway discharge is less than the normal outlet discharge of the reservoir operated as a simple retarding basin. Where the safety of the structure is not involved, increase in costs must be considered relative to increased benefits obtained. For the hypothetical 1932 flood as shown on Plate 28, it is true that approximately 60,000 acre-feet are available in Everett at the time spillway discharge starts. Only 34,000 acre-feet are available 8 hours later at maximum spillway discharge, but prior to the second and larger spillway discharge there is no longer any but surcharge storage available in Everett and in Hopkinton. The first spillway discharge, with all outlets closed, is but 2,000 c.f.s. and volume about 3,400 acre-feet. The second spillway discharge is 3,700 c.f.s. plus 5,000 c.f.s. from the outlets. The first spillway discharge, both in peak and volume, is believed a satisfactory condition considering the magnitude of the flood, and the entire storage volume is utilized in control of the flood. In order to eliminate the first spillway discharge, it has been determined that an increase of 20% in canal capacity would be required, at an estimated increase in cost of at least $300,000. The hydrographs are shown on Plate 63. It is of interest to note that when the 1932 hypothetical flood inflow was corrected for the effect of the proposed Mountain Brook and West Peterboro Reservoirs, it was passed through the reservoir without the first spillway discharge and complete utilization of storage at the time of spillway discharge.

12. Effect of 50% Increase in Canal Capacity.- (a) The effect of the proposed canal and one with a capacity increased 50% on a flood greater than the reservoir design flood is shown on Plate 64. A single peak inflow flood similar in shape to the 1938 flood, but with a peak of 40,000 c.f.s., is used for illustration. A single peak flood is used in preference to a double peak, for it emphasizes more clearly the point raised in the memorandum referred to in paragraph 8, namely: that some spillway discharge occurs at Hopkinton while there is still available storage in Everett. With the present canal, spillway discharge occurs during the third day with a maximum discharge of 7,000 c.f.s. At this time (6 P.M.), there are 27,000 acre-feet of storage still available in Everett. However, for a flood of greater magnitude than the reservoir design flood, spillway discharge will always occur, regardless of the size and capacity of the canal. Consequently, if spillway discharge is certain to take place, it is desirable to have it extend over a longer duration with the lower peak discharge rather than over a short-time period with the higher discharge. This is illustrated on Plate 64, which shows that when the canal capacity is increased 50%, no spillway discharge occurs on the third day, but on the fourth day it has a peak discharge 1,500 c.f.s. greater than with the smaller canal system.

(b) It is estimated that a 50% increase in canal capacity would cost about $700,000. The following differences may be summarized in connection with the particular flood shown on Plate 64. (It should be remembered that this theoretical flood has a peak inflow 25% greater than the reservoir design flood.)

	Present Canal	Canal + 50%
Max. canal discharge	34,000 c.f.s.	38,500 c.f.s.
Max. spillway discharge	25,000 c.f.s.	26,500 c.f.s.
Max. spillway discharge occurring with storage still available in Everett	7,000 c.f.s.	0 c.f.s.
Max. water surface elevation	417.2	417.4

13. Summary.- (a) Plate 65 shows the effect of increasing the canal capacity on the canal and spillway discharges over the entire range of floods from the reservoir design flood to the spillway design floods. These curves again emphasize the point that for floods greater than the reservoir design flood, spillway discharges will occur ultimately, regardless of the size and capacity of the canal, and the full storage is utilized. The effect in enlarging the canal is to reduce the first spillway discharge that takes place while storage is still available in

Everett, but to increase slightly the second peak. The curves illustrate that the first spillway discharge is always insignificant compared with the second spillway discharge and that the downstream damages would be about the same with either canal. It should be noted that the curves on Plate 65 are based on the 1938 flood, which is one particular type of flood. Being a flood of record, it is believed to be representative of the shape of the probable floods on the Contoocook River. Due to the magnitude of the natural valley storage on the Contoocook River, it is difficult to conceive of a very peaked flood. As a check, however, such a flood was routed and the results did not differ appreciably from those shown on Plate 65.

(b) The use of drum gates in the weir has been considered but their effectiveness found to be out of proportion to their cost. The weir controls the discharge of the canal for low flows, but as the canal flow approaches the maximum (28,000 c.f.s.), the long canal itself becomes the control and the presence or absence of the weir (or opening and closing gates in a weir) has little effect (see Plate 66).

14. Conclusions.- It is concluded that:

(a) The canal capacity selected in the project report is adequate for maximum effectiveness of flood control operation for all floods up to the reservoir design flood.

(b) For floods greater than the reservoir design flood, the elimination of spillway discharge at Hopkinton prior to complete utilization of Everett storage would require an increase of from 20% to 50% in canal capacity for material effect. Such increase would cost from $300,000 to $700,000 and would not yield a commensurate increase in benefits. For excessive and rare floods, increasing the canal capacity does not (and cannot) eliminate the resulting major spillway discharge. Hence, the damage from such large spillway discharge would be suffered regardless of increased canal capacity. The spillway discharge for any flood reasonably in excess of the reservoir design flood is normally delayed sufficiently to realize the major reductions in relation to downstream peak flows.

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX C

GEOLOGY, SOIL AND DESIGN DATA

Table of Contents

APPENDIX C

GEOLOGY, SOIL AND DESIGN DATA

Table of Contents (cont'd.)

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX C

GEOLOGY, SOIL AND DESIGN DATA

PART I. WEST HOPKINTON STRUCTURES

1. West Hopkinton Dam.-

a. Geology and Description of Dam Site.- The West Hopkinton dam site is located on the Contoocook River about 1,250 feet upstream from the existing dam at West Hopkinton, New Hampshire. (See Plate 47.) The river at the site flows almost due north in a deep, narrow, very steep-walled valley which it has excavated through a hill of glacial till. Both abutments and the valley bottom are composed almost entirely of this glacial till. The pre-glacial valley of the Contoocook River appears to lie immediately under the site and runs toward the northeast. Rock is very deeply buried over the entire site.

b. Field Exploration of Foundation.- The foundation exploration included the general geological study of the valley and the drilling of 3 holes with a total footage of 225 feet in overburden. (See Plate 47.) During exploration for borrow materials for the dam at the initially selected site below West Hopkinton, 4 test pits were excavated on the left abutment as shown on Plate 52, which reveal the character of the material in the upper region.

c. Overburden and Rock Conditions at Site.- The overburden in the abutments and valley bottom is a glacial till deposit consisting of very compact clayey, silty sand and gravel, which is somewhat variable but not stratified. Preliminary investigations indicate that the till in the lower region in the left abutment is slightly more sandy than in other regions. According to the geological and drilling information, rock is at least 70 feet below the valley floor. The classification of the overburden at drill holes D-14, D-15 and D-29, which is considered representative, is shown on Plate 49. Typical grain-size curves of the overburden are shown on Plate 54. In general, except for the more sandy deposit in the left abutment, the compact glacial till has a uniform coefficient of permeability probably approaching .001 x 10^{-4} cm. per second. The coefficient of permeability of the sandy till in the left abutment is estimated to be 0.01 x 10^{-4} cm. per second. In the valley bottom there are deposits of till consisting mainly of clayey silt and fine sand. Prior to final design, the materials in these deposits will be extensively investigated to determine their characteristics. It is believed, however, that the materials in all deposits of overburden at the dam site have ample shearing strengths to permit a normal design of the embankment and conduit.

2. Dike H-2.-

a. Geology and Description of Dike Site.- Dike H-2 is located about one-half mile southeast of the center of West Hopkinton. It extends west from Emerson Hill across the Elm Brook Valley as shown on Plate 47. The valley at this site is broad and flat and is deeply filled with variable glacial and aqueoglacial deposits. Bedrock is deeply buried except near the right abutment. The left abutment is a sand terrace surmounted by low, irregular kames.

b. Field Exploration of Foundation.- Exploration in the vicinity of Dike H-2 consists of (1) auger holes and test pits originally intended for the location of borrow materials for the embankment at the lower site (see Plate 52), (2) drill holes and seismic lines for the development of an alternate spillway site through the right abutment of the dike, and (3) drill holes and seismic lines for the exploration of the foundation of the dike. (See Plate 47.) Of this work, the exploration which is pertinent to the foundation of the dike consists of 3 drill holes, 2 test pits and 11 auger holes.

c. Overburden and Rock Conditions at Site.- The overburden in the left abutment and valley bottom is water-laid deposits of variable sands and silts. In general, in the valley bottom below the 385 surface contour, the overburden varies from uniform fine sand near the surface to uniform fine sand and silt at a depth of 15 feet. It is estimated that the coefficient of permeability of the overburden in this section is approximately 1 x 10^{-4} cm. per second, and from preliminary investigation, the overburden does not appear to be greatly stratified. On the left abutment and in the valley bottom near the right abutment the top overburden is stratified variable sands. The overburden on the right abutment is a sandy glacial till. It is estimated that the coefficient of permeability of the stratified sand and right abutment material varies from 1 to 100 x 10^{-4} cm. per second. Bedrock is more than 60 feet deep in the valley bottom and under the left abutment. It rises in the right abutment, outcropping at several places. The bedrock in general is a schist with some granite zones. Overburden and rock descriptions at drill holes in the vicinity are given on Plate 49. Typical grain-size curves of the overburden materials are shown on Plate 55.

3. Dike H-3.-

a. Geology and Description of Dike Site.- The site of Dike H-3 is located approximately 2 miles northeast of West Hopkinton. (See Plate 47.) The dike extends across a pre-glacial valley so that rock is only accessible near the westerly end. The overburden in the valley bottom and abutments is variable, consisting of glacial and aqueoglacial deposits of sands.

b. Field Exploration of Foundation.- The foundation exploration consists of a geological study of the site and the drilling of one hole, D-31. (See Plate 47.)

c. Overburden and Rock Conditions.- The overburden consists of stratified sands which have an estimated coefficient of permeability varying from 0.1 to 50 x 10^{-4} cm. per second. General classification of the overburden at D-31 is given on Plate 49. All overburden materials in the dike foundation have ample shearing strength for normal design.

4. Spillway.-

a. Field Exploration for Spillway.- Three alternate locations of the spillway on bedrock foundations are available in conjunction with the location of the dam above the village of West Hopkinton, all three of which have been explored, surveyed and estimated. One possible spillway site on the west slope of Emerson Hill, which forms the right abutment of Dike H-2, has been explored with 5 drill holes and 23 seismic lines. The extent and location of the pertinent exploration for this site are shown on Plate 47. A second possible spillway site on the ridge southeast of Emerson Hill was explored with 29 seismic lines (see Plate 47). For a spillway at this site, Dike H-3 would be replaced by a dike extending along the ridge and eastward to a suitable abutment as indicated by the outline designated as Dike H-1 on Plate 47. A third possible spillway site, which is the site selected and described herein, is located through the east slope of Emerson Hill, which forms the left abutment of Dike H-3. This area has been explored by geological reconnaissance and 7 drill holes (see Plates 42 and 47).

b. Overburden and Rock Conditions at Selected Spillway Site.- In general, the overburden at the selected spillway site is variable, silty and gravelly sand with boulders on the hillside and uniform, slightly silty sand in the valley bottom. Rock, as shown by the preliminary rock contours on Plate 42 prepared from the drilling data and observation of rock outcrops, slopes toward the northeast and is fairly deeply buried in the vicinity of the river. The rock is a granite, generally sound and suitable for structure foundations. Descriptions of the overburden and bedrock at drill holes D-16, D-18, D-19, D-20 and D-23, which are in the vicinity, are given on Plate 49. Typical grain-size curves of the overburden are shown on Plate 56.

5. Canal No. 1.-

a. Geology and Description of Site.- Canal No. 1, located through the right abutment just upstream from the West Hopkinton dam site, connects the Contoocook River and Elm Brook valleys, as shown on Plate 47. The ridge through which the canal will be cut is composed almost entirely of very compact, silty, glacial till.

b. Field Exploration of Site.- The overburden in and adjacent to the proposed canal area was explored by 2 drill holes as shown on Plate 47.

c. Overburden and Rock Conditions.- The overburden is a very compact, silty and gravelly sand. Typical grain-size curves are shown on Plate 57 (impervious material), and descriptions of the overburden at the drill holes are shown on Plate 49. Exploration indicates that bedrock lies below the proposed canal bottom. The overburden is very impervious and it is estimated that the material has an angle of internal friction of 30 degrees and a cohesion of 0.20 ton per square foot in the natural state. It is believed that the ground-water table lies at a great depth below the surface of the ridge.

6. Exploration for Materials and Summary of Distribution.-

a. A summary of the sources of materials for embankment construction and concrete aggregates and the distribution of these materials to the several structures of the West Hopkinton group are contained in Table 1.

TABLE 1 - MATERIALS FOR WEST HOPKINTON STRUCTURES

MATERIAL			DISTRIBUTION					
Source	Type	Total Exc. C.Y.	West Hopkinton Dam C.Y.	Dike H-2 C.Y.	Dike H-3 C.Y.	Canal No. 1 C.Y.	Spillway C.Y.	Waste C.Y.
Canal No. 1	Impervious	1,440,000 (2)	150,000	740,000	240,000			310,000 (2)
Spillway	Pervious	444,000			430,000			14,000
	Rock (1)	425,000	36,000	72,000	165,000	91,000	4,000	57,000
Borrow Areas	Pervious	252,000	52,000	200,000				
	Sand & Gravel			31,000	26,000	42,000		
	Filter Gravel		4,000	19,000	29,000		500	
	Processed Sand & Gravel (for aggregates)		12,300				31,800	

Note: All earth quantities given in borrow pit measure.

(1) Loose measure (40% swell).
(2) Includes stripping.

b. The exploration for Canal No. 1, the excavation from which provides the entire source for impervious fill in the embankments in the vicinity of West Hopkinton, is described in paragraph 5. The material from this source is a silty and gravelly sand estimated to have a coefficient of permeability of $.001 \times 10^{-4}$ cm. per second, an angle of internal friction of 28 degrees, and a cohesion of 0.20 ton per square foot when compacted in the embankments. Typical grain size curves of this material are shown on Plate 57.

c. The exploration for the spillway from which all rock fill for the West Hopkinton structures and pervious fill for Dike H-3 will be obtained is described in paragraph 4. The pervious material from this source is variable, ranging from fine to medium sand to a very silty, slightly gravelly sand. Typical grain-size curves are shown on Plate 56. It is estimated that the material will have a coefficient of permeability of approximately 1×10^{-4} cm. per second and an angle of internal friction of 36 degrees when compacted in the embankment. Exploration for pervious borrow to supplement material obtained from the spillway cut for use in Dike H-3 was performed by excavating and sampling 5 test pits in the vicinity of the east abutment of Dike H-3 as shown on Plate 52 prior to the discovery that the quantity available from the spillway was both suitable and sufficient.

d. The exploration of borrow areas, to supply the remaining pervious fill requirements of the West Hopkinton structures and the entire requirements for sand and gravel fill, gravel filters and concrete aggregates for these structures, is shown on Plate 52. These borrow areas were located and explored in connection with previous investigations for materials for the dam at the lower site, using a total of 72 test pits and 74 auger holes, of which 67 test pits and 7 auger holes remain pertinent for the location of these materials for the upper site. The pervious material available from this source is a well-graded fine to coarse sand with a minor gravel content, typical grain-size curves of which are shown on Plate 57. The results of preliminary tests indicate that the material in a compacted state will have a coefficient of permeability of approximately 200×10^{-4} cm. per second and an angle of internal friction of 36 degrees. The deposit from which filter and aggregate materials will be obtained contains well-graded, clean sand and gravel which averages about 40 percent by weight greater than 1/4 inch according to field screening tests. The material passing the 1/4-inch sieve contains less than 5 percent silt.

7. Preliminary Design of Structures.-

a. West Hopkinton Dam.- This dam (see Plates 40 and 41) is located on the main river and consists of an earth embankment retaining section and twin conduit flood control outlets. The entire dam is founded on the compact and impervious glacial till overburden of the site, bedrock being buried at inaccessible depths. The embankment has a crest length of 670 feet, a maximum height of 75 feet, and a gross volume of 240,000 cubic yards of all materials. The embankment contains a large central compacted impervious fill section which is extended into the valley bottom and abutments in a trench to form an impervious seal with the foundation. The impervious section is flanked upstream and downstream with pervious fill to provide stability of the slopes and control of the seepage. The entire upstream slope and the portion of the downstream slope below El. 390 are protected against wave action, rain wash and eddies with dumped rock fill backed by a layer of gravel. The downstream slope above El. 390 is provided with a covering of sod. Materials for the embankment will be obtained from the sources indicated in Table 1. The outlet works are benched in the till of the right abutment and consist of an intake structure, twin semi-circular roofed conduits, and stilling basin. Each conduit is 11 feet wide by 13 feet high and is controlled by two 7 x 10-foot gates. The outlet structures are of reinforced concrete throughout.

b. Dike H-2.- The preliminary design of Dike H-2, based on limited exploration of the foundation and available borrow materials, is shown on Plates 40 and 41. The dike will have a top length of 3600 feet, a maximum height of 70 feet, and a gross volume of 986,000 cubic yards of all materials. The main body of the embankment will be constructed of rolled impervious fill which will be extended into the foundation in a trench a nominal distance of 5 feet. A blanket of the same material will be extended from the impervious fill of the embankment a distance upstream sufficient to provide safe reduction and control of the seepage through the foundation. The impervious fill section of the embankment will be flanked both upstream and downstream by rolled pervious fill sections to add stability to the slopes. A drainage trench excavated into the foundation to a depth of 10 feet and backfilled with sand and gravel will be constructed at the downstream toe of the embankment to provide a safe outlet for seepage. The upstream slope will be protected from wave action by rock fill placed to a depth of 5 feet normal to the slope and backed by a 1-foot layer of gravel. The downstream slope will be covered with sod, except that below El. 390 the toe of the embankment will be protected with rock fill. Materials required for the construction of the embankment will be obtained from the sources indicated in Table 1.

c. Dike H-3.- The preliminary design of Dike H-3, based on limited exploration of the foundation and materials available, is shown on Plate 44. The dike will have a top length of 3900 feet, a maximum height of 63 feet, and a gross volume of 840,000 cubic yards of all materials. The embankment contains a narrow central rolled impervious fill section which will be extended into the foundation in a trench a nominal distance of 5 feet. A blanket of the same material will be extended upstream from the impervious core a distance sufficient to provide safe reduction and control of the seepage through the foundation. The impervious core will be flanked both upstream and downstream with rolled pervious fill sections, which in turn will be flanked by outer rock-fill sections which will be separated from the pervious fill by a 2-foot normal thickness of sand and gravel. A drainage trench under the downstream toe will be excavated to a depth of 12 feet into the foundation and backfilled with filters and rock fill to provide a safe outlet for foundation seepage.

d. Spillway.- As outlined in paragraph 4a of this appendix, exploration, surveys and estimates have been made of three alternate spillways in conjunction with the location of the West Hopkinton Dam at the upper site. Cost estimates of the several layouts, including costs of other factors affected by the alternate locations, indicated a close agreement between all three sites. The site selected offers the safest solution to the passing of floodwater because of the remote location of the discharge channel with reference to adjacent earth structures. It avoids, in addition, the necessity of excavating a long discharge channel to return the flow to the natural river channel and, further, it offers the best opportunity for the development of power at the site should future requirements indicate a power installation to be desirable. The spillway will consist of a channel 300 feet wide cut in the overburden and bedrock of the hill forming the left abutment of Dike H-3 (see Plate 42). The average length of the channel from the control weir to the end of the stilling basin will be about 1200 feet. The control weir will be a concrete cap of Ogee shape constructed on the bedrock floor of the channel, will be placed at an angle of approximately 30 degrees with the axis of the channel, and will have a crest length of 650 feet. A secondary concrete weir normal to the axis of the channel will be constructed below the main control weir to provide the depth of tailwater in the upper channel section required for satisfactory hydraulic action of the spillway to produce a reasonably uniform distribution of flow in the channel below. Concrete gravity retaining walls will be constructed along the sides of the channel where and to the heights required to retain the depth of flow for the maximum design discharge condition.

e. Canal No. 1.- This canal consists of a channel cut through the impervious glacial till ridge separating the Contoocook River and Elm Brook valleys (see Plates 40 and 41). The maximum depth of cut to the canal floor at El. 380 is 110 feet. The bottom width of the canal varies from 120 feet at the entrance to 150 feet at a distance of approximately 1130 feet from the entrance, from which point the width decreases in a distance of 920 feet to 50 feet which is maintained to its termination near Elm Brook. The initial section of the canal, 1130 feet long, passing through the region of heavy cut, will be protected against scour and sloughing with a blanket consisting of 3 feet of dumped rock backed with 1'-6" of sand and gravel which will be placed on the floor and side slopes of this section up to El. 430 (slightly higher than the maximum water surface of the spillway design flood). The side slopes of the canal will be cut to 1 on 2-1/2 below El. 430 and 1 on 2 above, which are believed to be satisfactory in view of the rock fill protection provided and the probable low natural level of ground water in the glacial deposit through which the canal will be cut.

8. Geology and Description of Site.- The canal, located approximately 4 miles southeast of West Hopkinton, N. H., connects the valleys of Elm Brook and Choate Brook as shown on Plates 2 and 43. The headwaters of Elm Brook, which is a tributary of the Contoocook River, are separated from Choate Brook by a low marshy divide. Choate Brook is a small tributary to the Piscataquog River. In general the brooks are sluggish streams flowing between low banks in broad marshy valleys. Choate Brook is somewhat constricted in the region near Sugar Hill. Irregularly elongated bedrock hills rising 100 to 250 feet above the valley floors bound the valleys on both sides and outcrops are abundant on the hillsides.

9. Exploration.- The area through which the proposed canal is located has received a general geological study from reconnaissance data and has been investigated in detail with 89 seismic lines, 14 drill holes and 52 probings. The seismic lines and drill holes were used to determine the surface of the bedrock. Auger holes were used to trace the limits of muck areas. The location of the drill holes is shown on Plate 43.

10. Rock Conditions.- From outcrops and drill hole data, the rock appears to be a schist with considerable pegmatite or granite areas. Description of the bedrock at drill holes D-1 to D-14, inclusive, which are located in the canal area are given on Plate 50. A bedrock contour map which has been used for estimates of rock excavation was prepared from the data obtained from seismic and drill hole exploration. A profile of the rock surface on the centerline of the canal is shown on Plate 43.

11. Overburden.- Data obtained from drill holes, augering and seismic investigations show that the overburden in the canal area is quite variable. In general, the overburden is a silty and gravelly sand with probable numerous localized cobbles and boulders. In the areas between Stations 15+00 and 32+00 and near Station 75+00 (see Plate 43), the top overburden consists of unstable soft muck which extends to depths varying from 3 to 25 feet. Descriptions of the overburden at the drill holes are given in Plate 50.

12. Design and Description.- Canal No. 2 will have a total length of 14,500 and will require the excavation of 1,500,000 cubic yards of overburden and 84,000 cubic yards of rock. The bottom width of the canal will be 170 feet. The sides will be excavated to a slope of 1 on 2-1/2 for the greater length of the canal where stable materials occur and to a slope of 1 on 5

where the overburden has been found to be unstable. A concrete gravity weir, founded on bedrock and having a crest elevation of 400, a net crest length of 300 feet and a maximum height of 32 feet will be constructed across the canal at a point approximately 2000 feet from the Everett end. The weir will serve to reduce the frequency of passing flood water to the Everett side of the reservoir, control velocities in the canal under all discharges to a maximum of 6 feet per second and, in the future, enable the conversion of a limited amount of storage in the West Hopkinton Basin to conservation use should such multiple-purpose operation of the reservoir be found desirable. The side slopes of the canal below the weir will be covered with 3 feet of dumped rock as a protection against serious erosion from possible high velocities.

PART III. EVERETT DAM

13. Geology and Description of Dam Site.- The Everett Dam Site is located approximately 1-1/4 miles southeast of East Weare, N. H., on the Piscataquog River (see Plate 2). At the site the river flows in a narrow valley, the right side of which is a precipitous rock cliff and the left, a moderately steep slope with almost continuous rock outcrops. The overburden in the valley bottom is generally less than 30 feet in depth and is a sand and gravel deposited by the glacier and by swollen and debris laden streams from the Contoocook Valley. At the base of the precipitous cliff there are boulders which have avalanched from the higher slopes.

14. Field Exploration of Foundation.- The foundation exploration completed to date includes the drilling of 9 holes with a total footage of 180 feet in overburden and 245 feet in rock and the determination of rock elevations by 24 seismic lines. The location and extent of the exploration are shown on Plate 48. Three drill holes were pressure tested to maximum depths in rock ranging from 30 to 50 feet.

15. Overburden Conditions at Site.- The overburden over the left abutment is generally shallow consisting of variable gravelly sand and in the valley bottom is generally less than 30 feet deep consisting of pockets, lenses and strata of variable sands and gravels. The overburden on the right abutment which is about 15 feet deep contains numerous large boulders which have avalanched from the higher slopes. The classification of the overburden at each drill hole is shown on Plate 51. Typical grain size curves of the overburden in the valley bottom and on the right abutment are shown on Plate 58. According to the preliminary tests the overburden in the valley bottom has a coefficient of permeability ranging between 1 and 25 x 10^{-4} cm. per sec.

16. Rock Conditions at Site.- Many rock outcrops occur at the dam site and its vicinity. The preliminary rock contours shown on Plate 48 and the centerline profile on Plate 45 were developed from rock outcrops and the results of drilling and seismic investigations. The bedrock of the left abutment slopes westward and meets the right abutment rock at a depth of 30 feet near the west side of the valley. Rock is generally exposed or close to the surface on the left abutment in the proposed area for the spillway. Outcrops extend on the left abutment for several hundred feet south of the proposed centerline beyond which the rock slopes southward. The bedrock of the right abutment follows the slope approximately 15 feet below

ground surface. Around elevation 420 on the right abutment there is a shelf about 100 feet wide as shown on Plate 48 above which rises the precipitous ledge cliff. The bedrock, as evidenced by the rock cores and outcrops, is in general a schist with occasional small granite and felsite dikes. Contacting the schist on the right abutment around elevation 440 is a coarse porphyritic gneiss. The bedrock along the centerline including the right abutment is structurally sound with only minor weathering and occasional fractures and joints. Downstream in the vicinity of drill hole D9 in the outlet discharge channel the rock is badly fractured and brecciated along several probably ancient shear zones. Recementation with calcite has occurred in these shear zones. According to the pressure pumping tests, the rock at the centerline contains only very minor fissures and seams. The maximum loss of water was about 1.3 gallons per minute under a pressure of 25 pounds per square inch.

17. Materials for Embankment and Concrete Aggregates.- (a) Exploration.- Exploration to locate suitable materials for the embankment and for concrete aggregates includes the excavation and sampling of 89 test pits and auger holes with a total of 641 feet of excavation and 175 feet of augering. One hole was drilled to a depth of 31 feet in the borrow area for impervious material. The location and extent of the exploration are shown on Plate 53. It is believed that ample quantities of satisfactory materials for the construction of the embankment and the production of concrete aggregates are available in the borrow areas explored in combination with material from structure excavation at the damsite.

(b) Pervious Material.- Material for the compacted pervious fill sections of the embankment is available from the pervious borrow area (Plate 53) adjacent to the site and is a gravelly fine to coarse sand. Typical grain size curves of the material are shown on Plate 59. According to the preliminary tests the material when compacted has a coefficient of permeability of about 400×10^{-4} cm. per sec. and an angle of internal friction of 36 degrees. Material from the proposed structure excavation areas is a well graded gravelly sand, suitable for fill in the pervious sections of the embankment.

(c) Impervious and Selected Impervious Materials.- Impervious material is available from the borrow area located approximately 1-3/4 miles from the site as shown on Plate 53. In general, the borrow area deposit contains a well graded very silty sand till. Below this sandy till, it is believed that more impervious material exists to a limited extent. This material, termed "selected impervious", is a well graded sandy silt with about 5% clay sizes. Typical grain size curves of the impervious and selected impervious materials are shown on Plate 60. From the preliminary permeability tests the coefficient of permeability of the impervious and selected impervious materials are 0.1 and 0.001×10^{-4} cm. per sec., respectively. Preliminary tests show that the selected impervious material has a shearing strength equivalent to approximately a cohesion of 0.1 ton per square foot and an angle of internal friction of 25 degrees. The preliminary tests showed that the impervious material has an angle of internal friction of approximately 36 degrees and no appreciable cohesion.

(d) Sand and Gravel for Filters and Backing.- The sand and gravel materials for filters and backing are available from the borrow area for pervious material. The material in the deposit is a well graded clean sand and gravel which, according to the field screening tests, averages about 60 percent by weight of gravel sizes greater than 1/4 inch.

(e) Rock.- Rock which is suitable for rock fill and riprap is available from the proposed structure excavation and is, in general, a schist as described in paragraph 16.

18. Selection of Type and Arrangement of Structures.- (a) Embankment.- It is proposed to construct an earth embankment, to serve as the main retaining section of the Everett Dam, having a length from the right abutment to the spillway of 1250 feet and a maximum height of 115 feet (see Plates 45 and 46) using the materials described in paragraph 17. The embankment will contain a central compacted impervious fill section which will extend to rock for the entire length of the embankment. A concrete cutoff wall founded on the uncovered bedrock surface will be constructed in the narrow trench section of the core to afford greater impermeability to percolation. However, if additional exploration discloses a sufficient quantity of the material designated as "selected impervious" in paragraph 17c, consideration will be given to the use of this material in the trench which, because of its greater impermeability, will eliminate the need for a concrete barrier. The compacted impervious core will be flanked with compacted pervious fill sections which rest on the foundation overburden materials and in localized areas on bedrock. A dumped rock fill section backed with a layer of sand and gravel will be placed on the upstream slope to protect the embankment during reservoir drawdown and from wave action. Since the foundation overburden and the material in the downstream section are relatively pervious compared to the core material, no drainage features in the downstream section will be required for the control of seepage. A small downstream rock toe will be provided to protect the embankment against any possible toe erosion.

(b) Spillway and Outlets.- The spillway will be located on the broad bedrock exposure of the left abutment and will be a low-height gravity structure having an overall length between retaining walls flanking the adjoining embankment sections of 212 feet and a net width, deducting the two piers of the access bridge, of 200 feet. Flood control regulation will be provided by outlet works entirely founded on bedrock and extending through the embankment approximately midway between the spillway and the present river channel. The outlet works will consist of an approach channel, a reinforced concrete intake structure and two rectangular conduits 7 feet wide by 8 feet 6 inches high, which will extend through the embankment for a total distance of 390 feet to a conventional type concrete stilling basin. Each conduit will be controlled by a 7 x 9 foot service and emergency gate contained in the intake structure. The outlet discharge channel will join the river approximately 360 feet below the toe of the embankment.

(c) Economy of Construction.- The arrangement and type of structures described above are believed to be the most economical for the site because of (1) the accessible and favorable location of bedrock on the left abutment for the construction of a spillway and spillway discharge channel and the construction of the outlet works, (2) the suitability of earth and rock materials obtained from excavation for these structures for use in the embankment, and (3) the availability of satisfactory borrow materials in nearby borrow areas and the suitability of the foundation for the safe construction of an earth embankment. The cost of an all-concrete gravity dam would exceed considerably the cost of the structures proposed.

PART IV. DIKES P1 AND P2

19. Geology and Description of Sites.- Dikes P1 and P2 are located near two divides between the valleys of Stark Brook and Bela Brook approximately 8 miles southeast of West Hopkinton, N. H., near Pages Corner, as shown on Plate 2. The saddle in which Dike P2 is located is a rock divide with shallow overburden of variable silty sand and gravel. The bedrock in general is a schist cut by quartz and granitic zones. Bedrock in the vicinity of the site for Dike P1 is not accessible and is deeply covered with variable deposits of stratified sands.

20. Exploration.- The sites and vicinity have been explored by geological reconnaissance to determine the foundation conditions and to locate suitable embankment materials. The depth of overburden along Dike P2 has been determined with 6 seismic lines. No test pits or drill holes have been employed in the exploration.

21. Preliminary Design of Embankments.- According to the geology of the region, it is believed that the foundations are satisfactory and that suitable embankment materials can be obtained readily. On this basis, the conventional designs shown on Plate 44 have been prepared for this report. Dike P1 will have a top length of 3550 feet, a maximum height of 48 feet, and will contain a gross volume of fill of 480,000 cubic yards. Corresponding figures for Dike P2 will be: Length, 2350 feet; height, 27 feet; and, fill, 120,000 cubic yards. According to the seismic information, the overburden along the centerline of Dike P2 varies from 2 to 12 feet in depth except under the right abutment, which will permit the construction of an economical cut-off to bedrock for the greater length of the embankment. The 3-foot blanket of dumped rock shown on the upstream slopes of both dikes and the material for hand-placed riprap at the downstream toes will be obtained from rock excavation in Canal No. 2. To avoid unnecessary interference with local developments Dike P1 has not been located where the minimum volume of fill is obtained. Preliminary studies indicate, however, that the location selected, considering all items of cost involved, is about economically equal to the location requiring the least volume of fill.

PART V - CONSTRUCTION PROCEDURE

22. General.- There are three general groups of construction operations involved in the Hopkinton-Everett project:

(a) West Hopkinton Structures (Dam, Dikes H-2 and H-3, Canal No. 1, and the spillway)

(b) Everett Dam

(c) Canal No. 2 and Dikes P-1 and P-2

These groups of operations can be carried on simultaneously or in sequence. The project, therefore, can be accomplished by means of one, two, or three separate contracts as desired. If the groups of operations are carried on in sequence, the West Hopkinton group must be completed first, and Everett Dam must be substantially completed before Canal No. 2 is cut through entirely.

23. West Hopkinton Structures.- Two seasons will be required for the construction of this group, one season before diversion of the stream and one following. The initial work will consist of the preparation of foundations for Dikes H-2 and H-3 and stripping of Canal No. 1, the spillway and borrow areas for pervious and sand and gravel materials. Provision for passing the flow of Elm Brook, which has a drainage area of 9 square miles, will be made during the first season of construction of Dike H-2, either by placing a culvert in the fill which will be subsequently sealed or by leaving a gap in the fill until excavation of Canal No. 1 has progressed to a depth sufficient to permit diversion of the brook to the Contoocook River. No provisions for diversion are required at Dike H-3, except possibly for the construction of a small dike across the valley bottom and early excavation of the drainage channel shown on Plate 47. The outlet works of the West Hopkinton Dam will be constructed in the latter part of the first season, during which time the present pond at the site will be drawn down and a low cofferdam placed in the river paralleling the outlet works construction area to enable operations to proceed in the dry. In the spring of the second season, an earth and rock fill cofferdam will be constructed across the river between the intake structure and left abutment for diversion of the river through the outlets, and a smaller downstream cofferdam will be placed between the stilling basin and left abutment to permit unwatering of the site. Completion of the excavation of Canal No. 1 and the spillway, and placing and compacting fill in the dam and two dikes, will be continued and completed in the second season.

24. <u>Everett Dam</u>.- The outlet works will be completed in full, except for upper lifts of the intake structure, prior to diversion of the river. In the prosecution of this work the construction of the conduits will be accomplished first, to enable direct placement of materials from the balance of the excavation for the outlet works in the portion of the embankment located over the conduits between the river and the spillway. The upstream and downstream cofferdams will be principally of rock fill, with outer impervious blankets, and will form a permanent part of the embankment, except possibly for the removal of the downstream impervious blanket. The upstream cofferdam will be 35 to 40 feet high, since the outlet invert will be approximately 15 feet above the present water level. Following construction of the cofferdams, the central core trench will be excavated to rock, the concrete cut-off wall placed, and the trench backfilled. Excavation of the spillway to supply pervious fill and rock, and working of the impervious borrow area, will be prosecuted as required for the construction of the embankment. Concrete aggregates will be processed from material available in a sand and gravel source adjacent to the dam site. Construction of Everett Dam will be completed in two seasons.

25. <u>Canal No. 2 and Dikes P-1 and P-2</u>.- The excavation for the canal will be initiated from either or both ends to provide drainage of the swampy canal area by gravity. The earth excavation will be deposited in waste areas bordering the canal. Occasional haul of earth will be required for the construction of spoil dikes across bordering flats to provide a uniform cross-section and for direction of canal flows. Areas behind these dikes will be drained by culverts placed in the fill. Rock excavation will be used for protection of the canal side slopes in earth, possibly for slope protection on the reservoir side of the dikes, and the balance wasted in spoil areas. Dikes P-1 and P-2 will be constructed of rolled earth fill, with reservoir slopes protected by rock fill obtained from the canal excavation. Excavation of Canal No. 2 and construction of Dikes P-1 and P-2 will be accomplished in two seasons.

DEFINITE PROJECT REPORT

HOPKINTON-EVERETT RESERVOIR

APPENDIX D

DETAILED ESTIMATE OF COST

I. LAND, RIGHTS-OF-WAY & RELOCATIONS

	Quantity	Unit Price	Total Cost
Land	7,450 acres	$ 22.25	$ 168,000
Buildings	lump sum	-	510,000
Cemetery relocation (2,880 graves)	lump sum	-	144,000
Power and telephone relocation .	lump sum	-	65,000
Water systems	lump sum	-	13,000
Legal expenses and surveys . . .	lump sum	-	120,000
Sub-Total			$1,020,000
Engineering, Overhead & Contingencies (25%±)			255,000
Sub-Total			$1,275,000
Highway relocation (21 miles).[(1)] .	lump sum	-	1,250,000
Railroad relocation (5.25 mi.)[(2)] .	lump sum	-	800,000
TOTAL - LAND, RIGHTS-OF-WAY & RELOCATIONS			$3,325,000

II. RESERVOIR CLEARING

	Quantity	Unit Price	Total Cost
Wooded area - complete clearing	370 acres	$ 125.00	$ 46,250
Wooded area - partial clearing .	3,510 acres	25.00	87,750
Cleared areas	2,330 acres	5.00	11,650
Sub-Total			$ 145,650
Engineering, Overhead & Contingencies (35%±)			50,350
TOTAL - RESERVOIR CLEARING			$ 196,000

(1) Estimate furnished by New Hampshire State Highway Department

(2) Estimate furnished by Boston & Maine Railroad Company

III. <u>WEST HOPKINTON DAM</u> (Top of dam El. 432; Spillway Elev. 412)

	Quantity	Unit Price	Total Cost
Stream diversion & pumping . . .	lump sum	-	$ 80,000
Clearing & grubbing dam site . .	lump sum	-	5,000
Stripping	32,000 c.y.	$ .50	16,000
Common excavation & haul	48,000 c.y.	.40	19,200
Impervious borrow	150,000 c.y.	(obtained from Canal #1)	
Pervious borrow	52,000 c.y.	.45	23,400
Structural backfill - compacted	23,000 c.y.	.40	9,200
Rolling impervious fill	138,000 c.y.	.12	16,560
Rolling pervious fill	47,000 c.y.	.12	5,640
Rock fill and dumped riprap . .	35,000 c.y.	.50	17,500
Riprap (hand-placed)	1,100 c.y.	2.50	2,800
Screened gravel	4,000 c.y.	2.00	8,000
Concrete - conduits	2,000 c.y.	16.00	32,000
" - walls	3,500 c.y.	16.00	56,000
" - intake structure and bridge pier	3,800 c.y.	18.00	68,400
Reinforcing steel	850,000 lbs.	.05	42,500
Trash bars (installed)	lump sum	-	8,000
Gates, guides, hoists & liners .	lump sum	-	50,000
Crane	lump sum	-	8,000
Operating house superstructure .	lump sum	-	25,000
Topsoil and seeding	lump sum	-	2,000
Access road	lump sum	-	10,000
Service bridge	lump sum	-	6,000
Miscellaneous items	lump sum	-	26,800
Sub-Total			$538,000
Engineering, Inspection, Overhead & Contingencies (35%±)			188,000
TOTAL CONSTRUCTION COST - WEST HOPKINTON DAM			$726,000

IV. EVERETT DAM (Top of dam Elev. 430; Spillway Elev. 414)

	Quantity	Unit Price	Total Cost
Stream diversion and pumping . . .	lump sum	-	$ 35,000
Clearing and grubbing dam site . .	lump sum	-	8,000
Stripping dam site	70,400 c.y.	$ 0.40	28,200
Common excavation and haul	162,000 c.y.	0.40	64,800
Rock excavation and haul	150,000 c.y.	2.00	300,000
Impervious borrow	205,000 c.y.	0.50	102,500
Pervious borrow	488,000 c.y.	0.35	170,800
Structural backfill - compacted .	4,100 c.y.	0.45	1,800
Rolling - impervious fill	190,200 c.y.	0.12	22,800
" - pervious fill	586,900 c.y.	0.12	70,400
Rock fill and dumped riprap . . .	195,200 c.y.	0.40	78,100
Sand and gravel backing	23,000 c.y.	1.20	27,600
Drilling holes for grouting . . .	8,900 l.f.	1.00	8,900
Pressure grouting	9,600 c.f.	1.25	12,000
Line drilling	11,400 s.f.	1.00	11,400
Hand-placed riprap	3,100 c.y.	2.50	7,800
Screened gravel	5,000 c.y.	2.00	10,000
Concrete - spillway, piers & walls	16,500 c.y.	13.00	214,500
" intake, gatehouse and conduits	6,300 c.y.	16.50	104,000
" stilling basin	4,400 c.y.	10.00	44,000
" cut-off wall	1,400 c.y.	16.00	22,400
Reinforcing steel.	900,000 lbs.	0.05	45,000
Approach bridge to gatehouse . . .	lump sum	-	10,000
Trash bars (installed)	lump sum	-	3,000
Gates, guides, hoists (installed) .	lump sum	-	30,000
Crane	lump sum	-	6,000
Operating house superstructure . .	lump sum	-	20,000
Access road, bridge, etc.	lump sum	-	20,000
Topsoil and seeding	lump sum	-	6,000
Improvements to Piscataquog River	lump sum	-	30,000
Miscellaneous items	lump sum	-	30,000
Sub-Total			$1,545,000
Engineering, Inspection, Overhead & Contingencies (35%±)			540,000
TOTAL CONSTRUCTION COST - EVERETT DAM			$2,085,000

			V		VI		VII	
			WEST HOPKINTON Spillway		WEST HOPKINTON Dike H-2		WEST HOPKINTON Dike H-3	
Item	Unit	Unit Price	Quantity	Total Cost	Quantity	Total Cost	Quantity	Total Cost
Clearing and grubbing	lump sum			$ 5,400		$ 12,000		$ 7,000
Stripping	c.y.	$ .40	80,000	32,000	71,000	28,400	66,000	26,400
Common excavation under dikes	c.y.	.40			40,000	16,000	66,000	26,400
Common excavation - spillway	c.y.	.30	444,000	133,200				
Rock excavation - spillway	c.y.	1.90	304,000	577,600				
Pervious borrow	c.y.	.40			200,000	80,000		
Rolling - pervious fill	c.y.	.12			184,000	22,080	400,000	48,000
Rolling - impervious fill	c.y.	.12			680,000	81,600	221,000	26,520
Structural backfill	c.y.	.40	5,000	2,000				
Sand & gravel toe & backing	c.y.	1.20			31,000	37,200	26,000	31,200
Screened gravel	c.y.	2.00	500	1,000	19,000	38,000	29,000	58,000
Dumped rock (placing only)	c.y.	.50	3,000	1,500				
Dumped rock (placing only)	c.y.	.45			72,000	32,400		
Dumped rock (placing only)	c.y.	.35					160,000	56,000
Hand-placed riprap	c.y.	2.50	1,100	2,750			3,400	8,500
Seeding and topsoil	lump sum			2,000		10,000		
Line drilling	s.f.	1.00	12,000	12,000				
Drilling holes for grouting	l.f.	1.00	2,500	2,500				
Pressure grouting	c.f.	1.25	2,500	3,120				
Concrete - spillway weir	c.y.	14.00	10,600	148,400				
Concrete - spillway retaining walls	c.y.	13.00	13,500	175,500				
Reinforcing steel	lbs.	.05	200,000	10,000				
Drainage canal above spillway	c.y.	.50	6,000	3,000				
Miscellaneous items				27,030		18,320		15,980
Sub-Total				$1,139,000		$376,000		$304,000
Engineering, Inspection, Overhead & Contingencies (35%±)				399,000		132,000		106,000
TOTAL CONSTRUCTION COST				$1,538,000		$508,000		$410,000

Item	Unit	Unit Price	VIII PAGES CORNER Dike P-1 Quantity	VIII PAGES CORNER Dike P-1 Total Cost	IX PAGES CORNER Dike P-2 Quantity	IX PAGES CORNER Dike P-2 Total Cost
Clearing and grubbing	lump sum			$ 7,000		$ 1,900
Stripping	c.y.	$ .50	45,000	22,500	19,000	9,500
Common excavation	c.y.	.40	14,000	5,600	6,400	2,560
Pervious borrow	c.y.	.40	275,000	110,000	95,000	38,000
Impervious borrow	c.y.	.40	162,000	64,800		
Rolling - pervious fill	c.y.	.12	255,000	30,600	89,000	10,680
Rolling - impervious fill	c.y.	.12	150,000	18,000		
Sand and gravel	c.y.	1.20	42,600	51,100	19,200	23,040
Dumped riprap (hauling and placing)	c.y.	1.00	27,000	27,000	11,100	11,100
Hand-placed riprap	c.y.	3.00	2,800	8,400	1,200	3,600
Seeding and topsoil	lump sum			5,000		1,500
Miscellaneous items				4,000		2,120
Sub-Total				$354,000		$104,000
Engineering, Inspection, Overhead & Contingencies (35%±)				124,000		36,000
TOTAL CONSTRUCTION COST				$478,000		$140,000

		X			XI		
		CANAL NO. 1			CANAL NO. 2		
Item	Unit	Quantity	Unit Price	Total Cost	Quantity	Unit Price	Total Cost
Clearing	acres	15.4	$ 130.00	$ 2,000	160	$ 130.00	$ 20,800
Common excavation	c.y.	1,440,000	.30	432,000	1,500,000	.30	450,000
Rock excavation	c.y.				84,000	2.15	180,600
Dumped riprap	c.y.	91,000	.70	63,700	50,000	.50	25,000
Concrete - weir	c.y.				4,500	12.00	54,000
Line drilling	s.f.				4,000	1.00	4,000
Drainage culverts	lump sum						15,000
Seeding and topsoil	acres	lump sum		11,000		lump sum	50,000
Sand and gravel backing	c.y.	42,000	1.30	54,600			
Miscellaneous items				11,700			28,600
Sub-Total				$575,000			$ 828,000
Engineering, Inspection, Overhead & Contingencies (35%±)				201,000			290,000
TOTAL CONSTRUCTION COST				$776,000			$1,118,000

SUMMARY

I.	Land, Rights-of-Way, and Relocations		$ 3,325,000
	Construction Costs		
II.	Reservoir Clearing	$ 196,000	
III.	W. Hopkinton Dam	726,000	
IV.	Everett Dam	2,085,000	
V.	W. Hopkinton Spillway	1,538,000	
VI.	Dike H-2	508,000	
VII.	Dike H-3	410,000	
VIII.	Dike P-1	478,000	
IX.	Dike P-2	140,000	
X.	Canal No. 1	776,000	
XI.	Canal No. 2	1,118,000	
	Sub-Total, Construction Costs		$ 7,975,000
	TOTAL ESTIMATED COST		$11,300,000
	Cost per acre-foot of total storage =		$72

COMPUTATION OF ANNUAL CARRYING CHARGES

Federal Investment

1.	Total First Cost	
	(a) Structures with 50-yr. life	$11,109,650
	(b) Equipment with 25-yr. life	$ 190,350
2.	Interest During Construction - 3% for 1-1/2 yrs.	
	(a) On structures with 50-yr. life	$ 500,350
	(b) On equipment with 25-yr. life	$ 8,650
3.	Total Investment	
	(a) Structures with 50-yr. life	$11,610,000
	(b) Equipment with 25-yr. life	199,000
	(c) Total	$11,809,000

Annual Charges

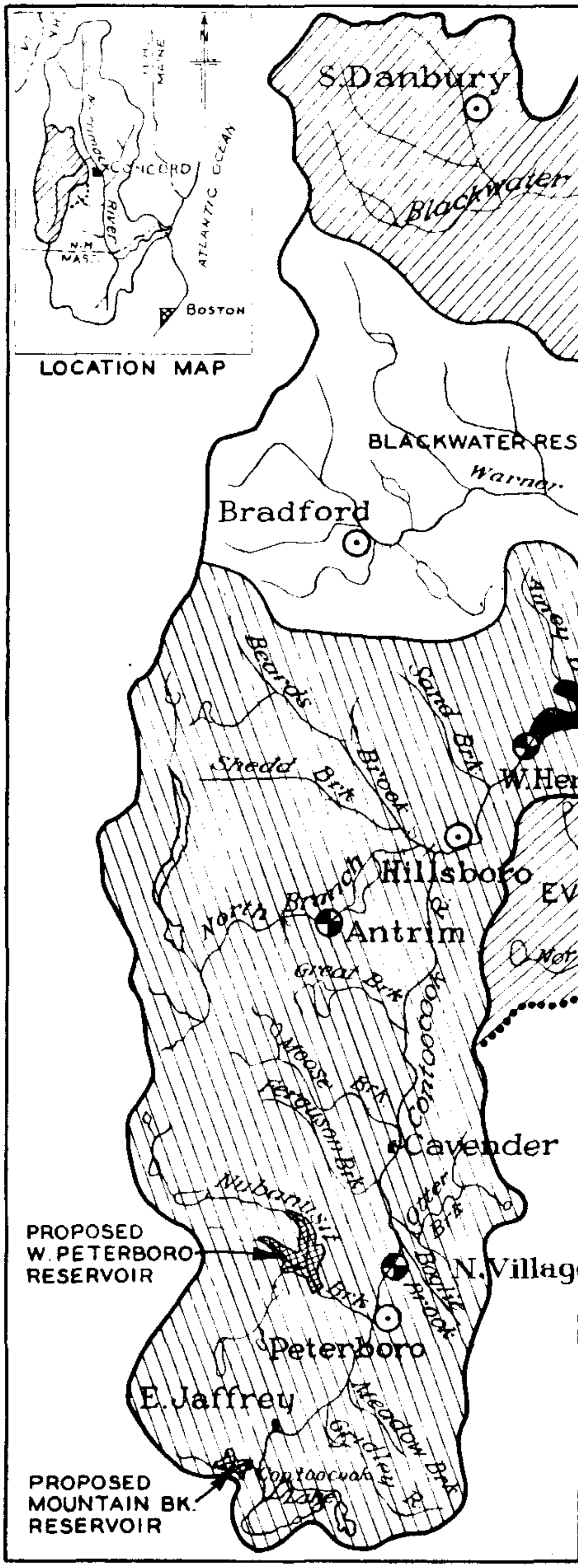
1.	Interest on Investment - 3-1/2%	$413,310
2.	Amortization	
	(a) Structures with 50-yr. life - .763%	88,580
	(b) Equipment with 25-yr. life - 2.567%	5,110
3.	Operation and Maintenance	10,000
4.	Total Annual Charges	$517,000

Construction Period - 3 Years.

List of Illustrations

List of Illustrations (continued)

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL

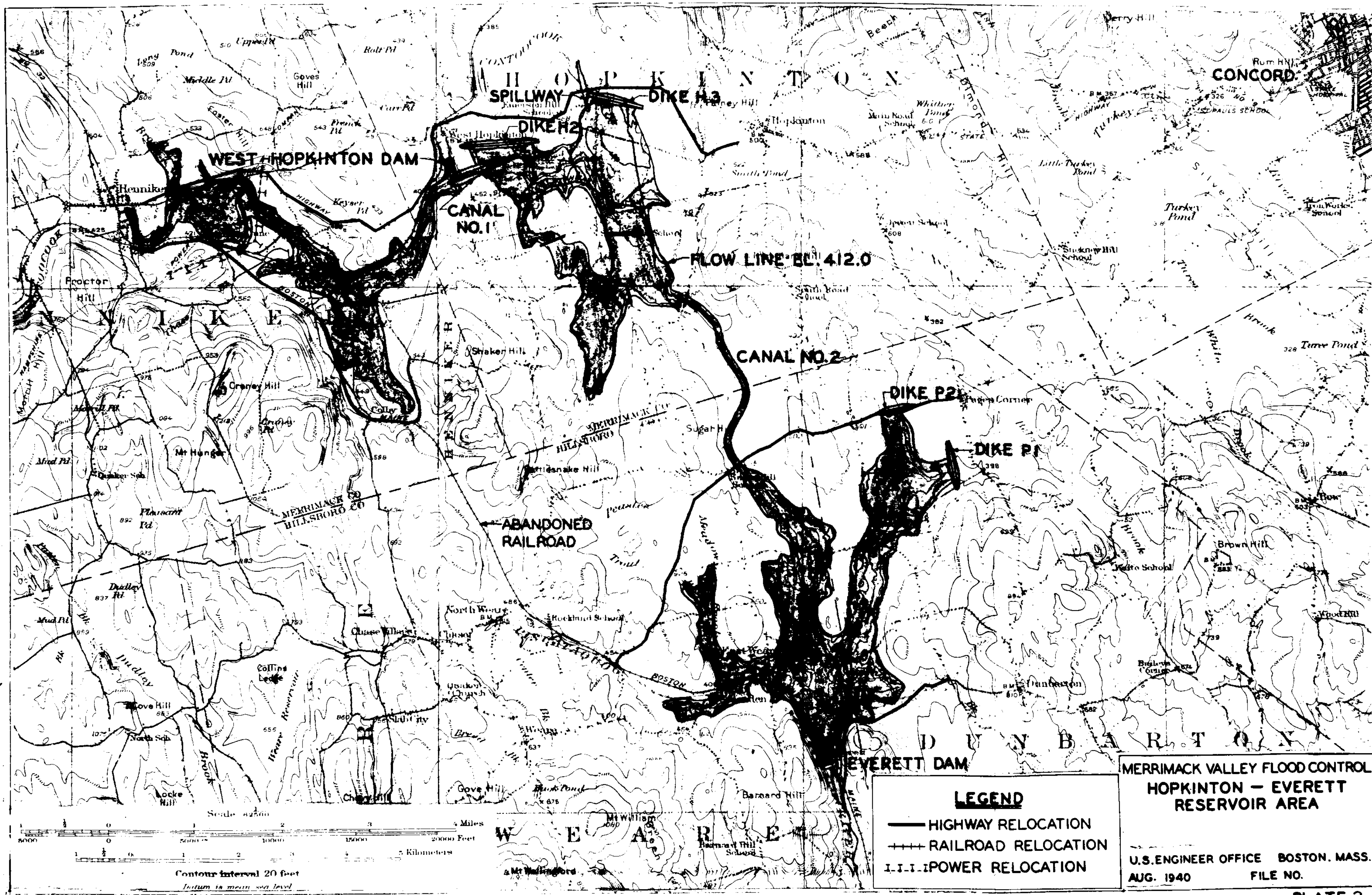
CONTOOCOOK AND PISCATAQUOG BASINS

PROPOSED HOPKINTON-EVERETT RESERVOIR

U.S. ENGINEER OFFICE BOSTON, MASS.

SEPT. 1940 FILE NO.

PLATE I



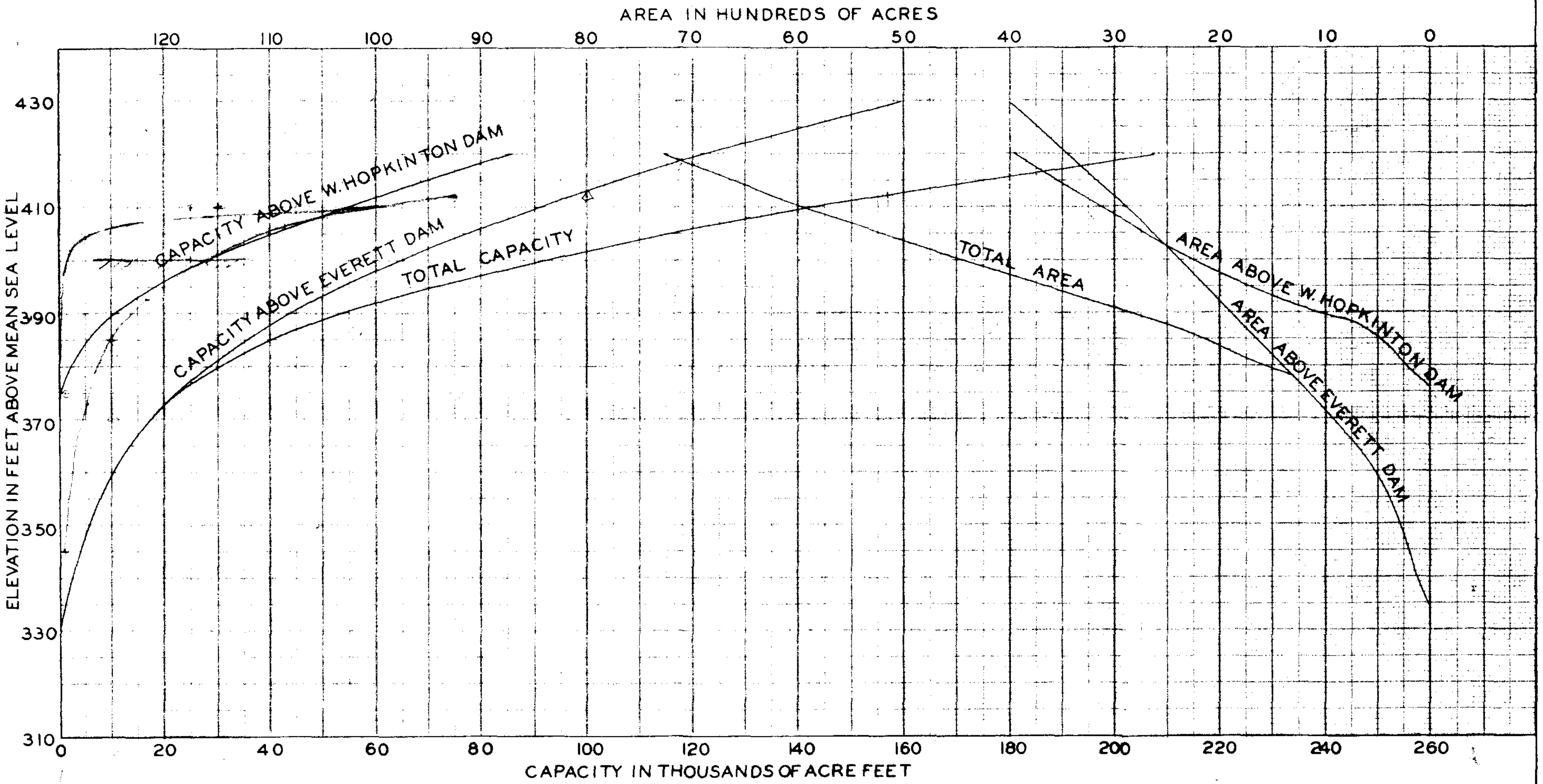
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON — EVERETT RESERVOIR AREA

U.S. ENGINEER OFFICE BOSTON, MASS.
AUG. 1940 FILE NO.

PLATE 2

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



MERRIMACK VALLEY FLOOD CONTROL

HOPKINTON-EVERETT RESERVOIR

AREA AND CAPACITY CURVES

U.S. ENGINEER OFFICE BOSTON, MASS.

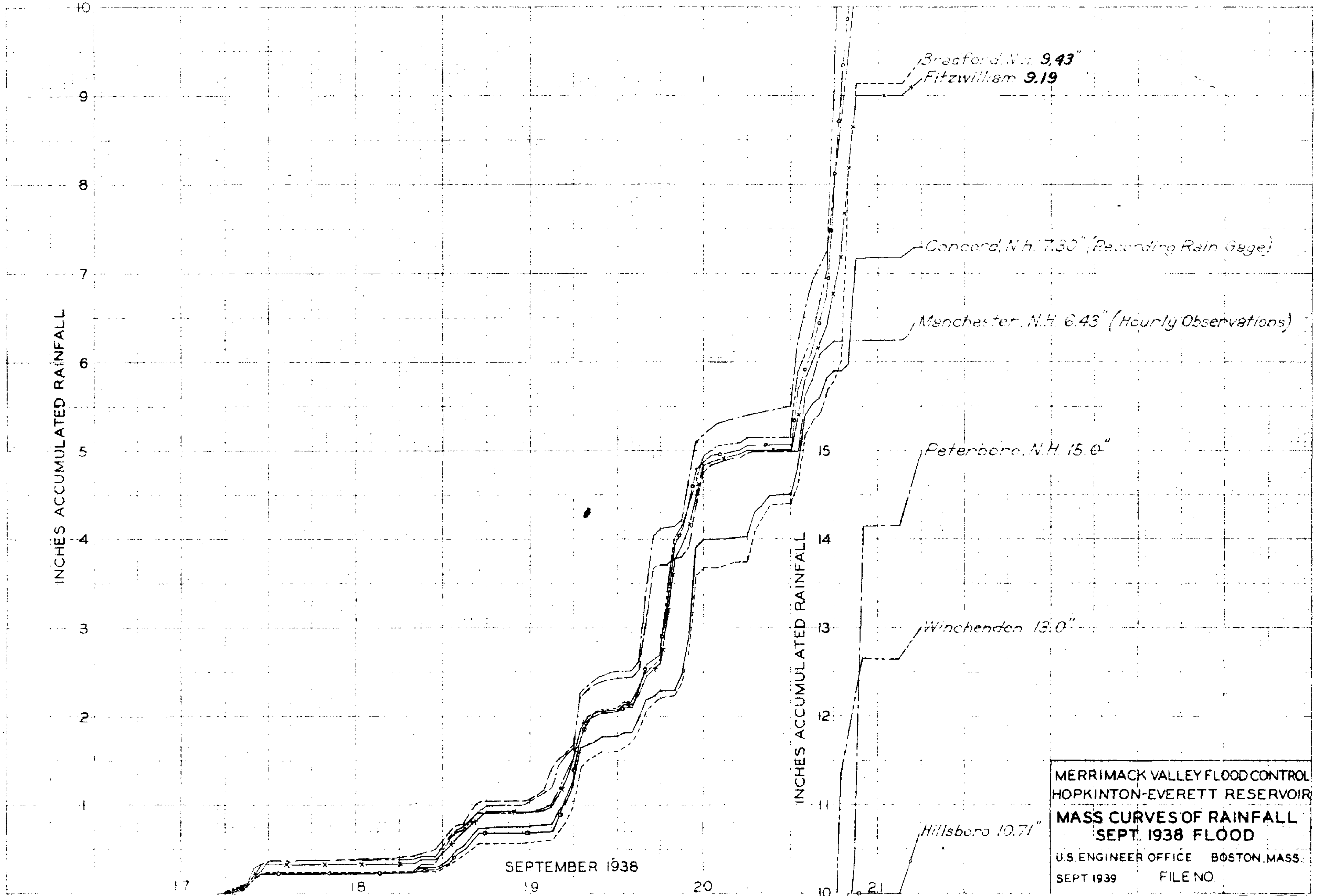
SEPT. 12 1940. FILE NO.

PLATE 3

ELEVATION IN FEET ABOVE M.S.L.
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
110
120
130
140
150
160
MILES ABOVE NEWBURYPORT LIGHT
HIGHLAND LAKE
ISLAND POND
SHEDD BROOK
NORTH BRANCH
JACKMAN RESERVOIR
AMEY BROOK
BEARDS BROOK
MOOSE BROOK
NUBANUSIT BROOK
SKATUTAKEE LAKE
CONTOOCOOK LAKE
OTTER LAKE
OTTER BROOK
TODDS POND
WARNER RIVER
BLACKWATER RIVER
WALKER POND
DEER MEADOW BROOK
PILLSBURY POND
CONTOOCOOK
RIVER
PENACOOK
RIVERHILL
BLACKWATER R.
WARNER RIVER
WEST HOPKINTON
AMEY BROOK
HENNIKER
WEST HENNIKER
GAGING STATION
P.S. CO. DAM
HILLSBORO
NORTH BRANCH
SHEDD BROOK
BEARDS BROOK
BENNINGTON
CAVENDER
NORTH VILLAGE, PETERBORO
PETERBORO
HADLEY
EAST JAFFREY
MERRIMACK VALLEY FLOOD CONTROL
PROFILES
CONTOOCOOK RIVER
AND
TRIBUTARIES
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.
PLATE 4


MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK AND
PISCATAQUOG BASINS
RAINFALL STUDIES
40,000'
0
40,000'
SCALE IN FT.
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939
FILE NO.
LEGEND
Areas for Thiessens Weighting
Outline of Drainage Areas
Location of Rainfall Stations
Location of Stream Gaging Stations
N.H.
MASS.
Winchendon
East Rindge
Rindge
Fitzwilliam
Troy
Marlboro
Dublin
Sharon
Harrisville
E Sullivan
Stoddard
S Stoddard
Marlow
Washington
E Washington
Lempster
Goshen
Mill Village
Newport
Sunapee
Georges Mills
Springfield
Enfield Cen
Lockehaven
E Lebanon
Etna
Hanover Cen
Grafton Cen
Grafton
Alexandria
Hebron
Groton
E Hebron
Newfound Lake
Bristol
Bridgewater
New Hampton
Ashland
Winona
Meredith
Meredith Cen
Center Harbor
Lakeview
Melvin
Long Island
The Weirs
Sanbornton
N Sanbornton
Winnisquam
Lakeport
Gilford
LACONIA
Loon Cove
Alton Bay
B E L K N A P
Franklin
Tilton
Belmont
Gilmanton
Northfield
E Canterbury
Canterbury
Suncook Ponds
Barnstead
Pittsfield
Boscawen
Loudon
N Chichester
M E R R I M A C K
Epsom
E Concord
CONCORD
Pembroke
Bow
Allenstown
Suncook
Dunbarton
Hooksett
Highlands
Candia
Auburn
Manchester
Goffstown
Francestown
S Weare
Weare
Hillsboro
Lower Hillsboro
Bradford
Sutton
Wilmot
Wilmot Flat
Andover
Webster
Penacook
CONTOOCOOK
BOSTON
Newfound
Lake
Winnisquam
Lake

PLATE 5

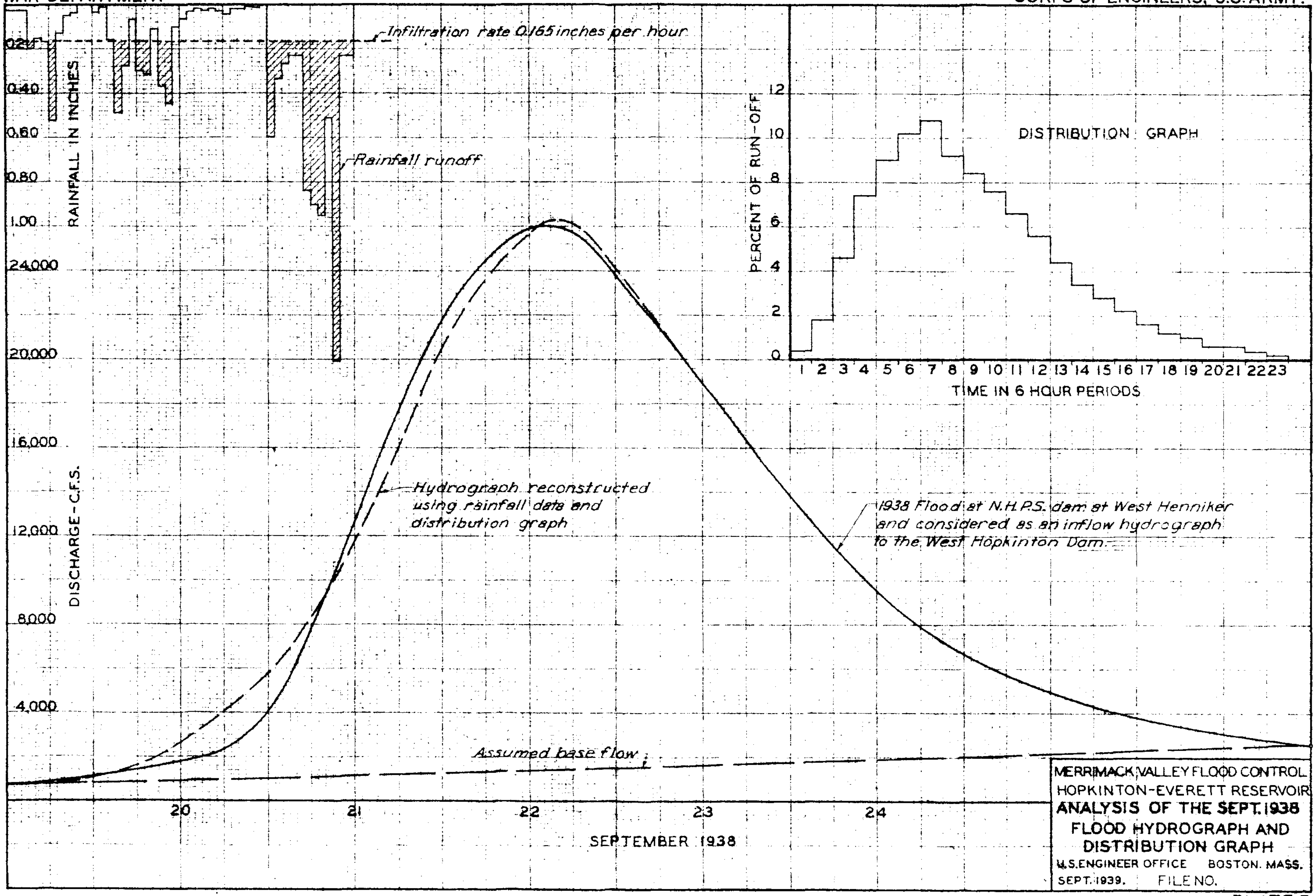


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

MASS CURVES OF RAINFALL
SEPT. 1938 FLOOD

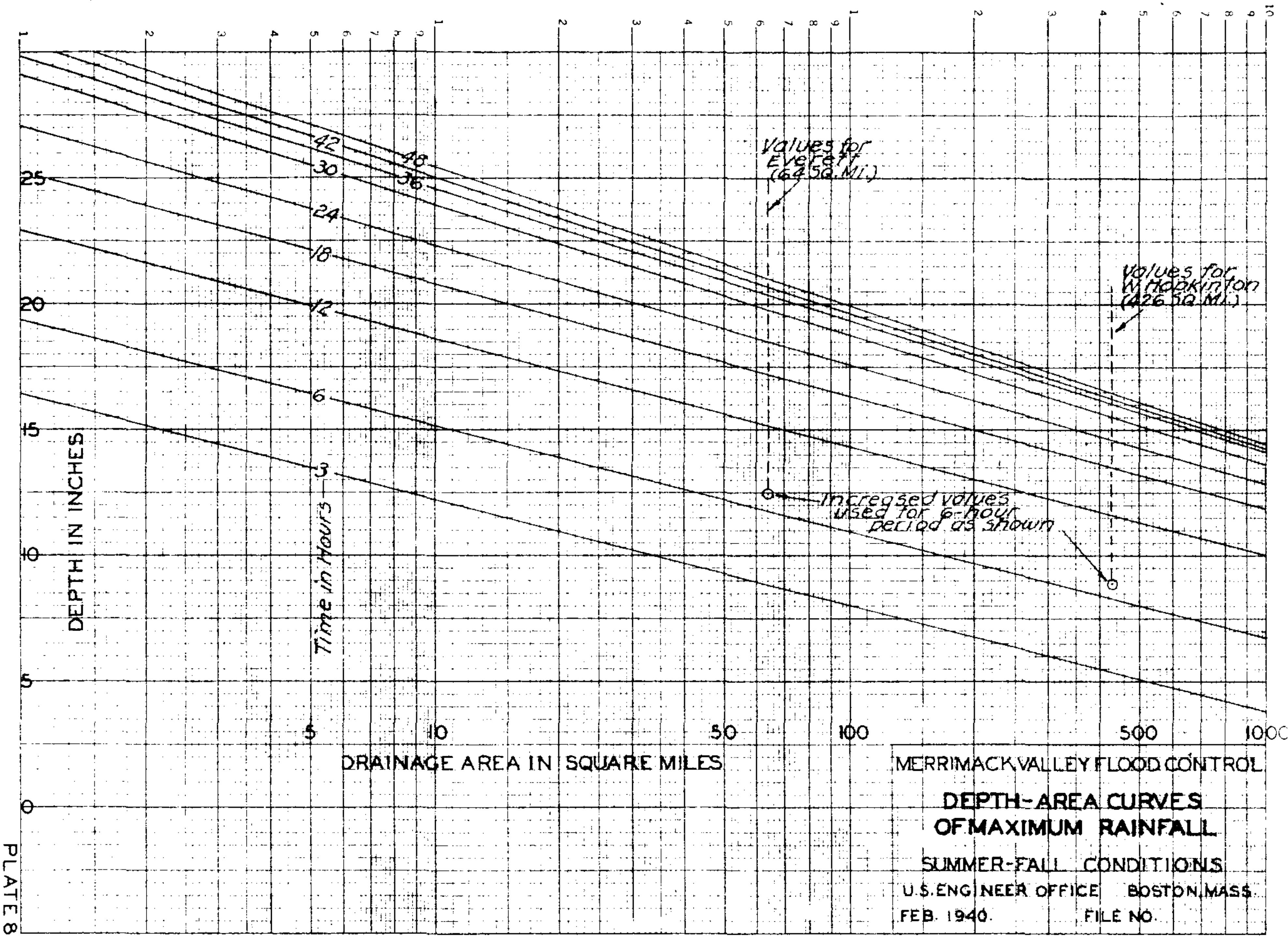
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT 1939 FILE NO.

PLATE 6



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
ANALYSIS OF THE SEPT. 1938 FLOOD HYDROGRAPH AND DISTRIBUTION GRAPH
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939. FILE NO.

PLATE 8


DEPTH IN INCHES
Time in Hours
DRAINAGE AREA IN SQUARE MILES
Values for Everett (64 SQ. MI.)
Values for W. Hopkinton (426 SQ. MI.)
Increased values used for 6-hour period as shown
MERRIMACK VALLEY FLOOD CONTROL
DEPTH-AREA CURVES OF MAXIMUM RAINFALL
SUMMER-FALL CONDITIONS
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

WAR DEPARTMENT

CORPS OF ENGINEERS U.S. ARMY

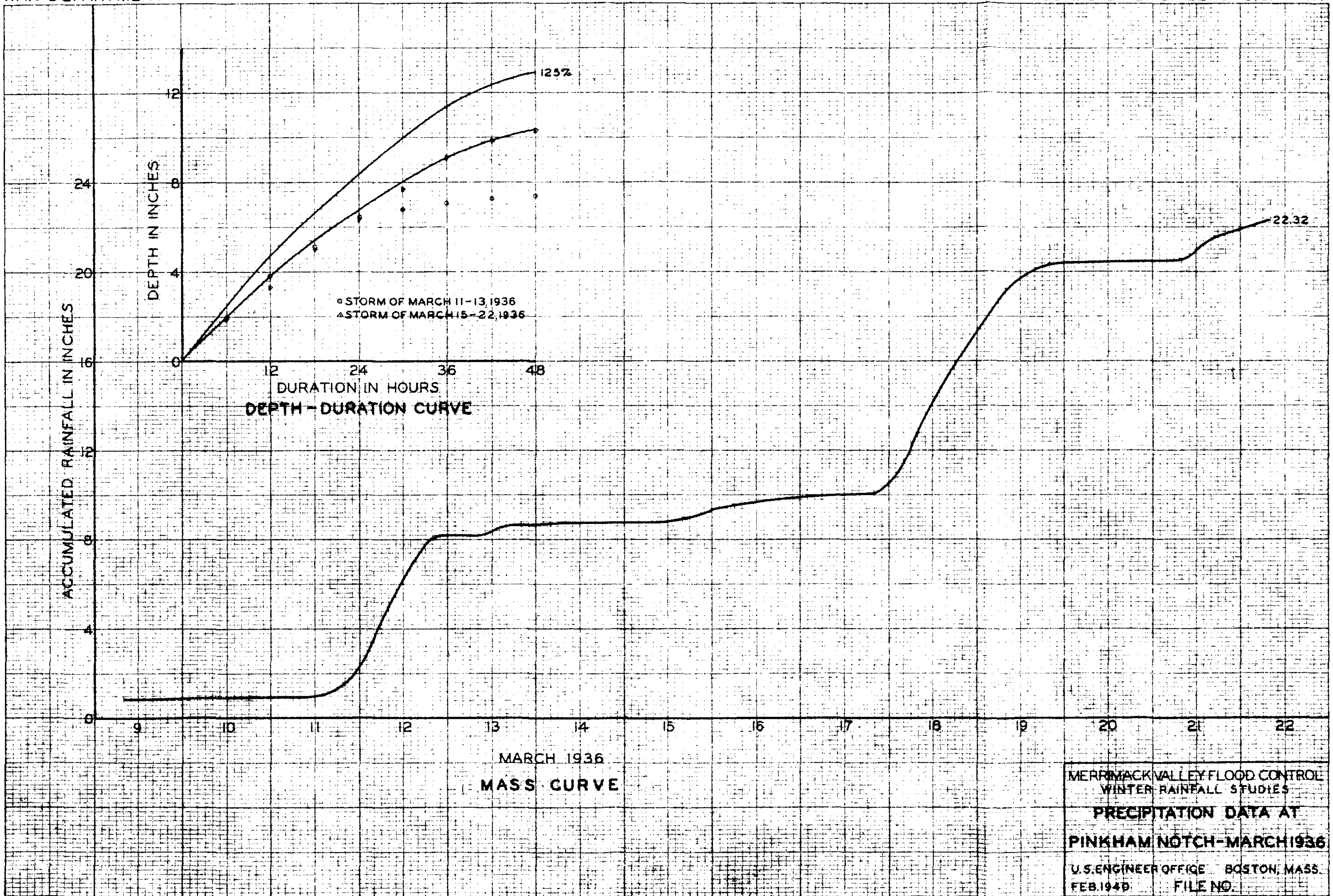
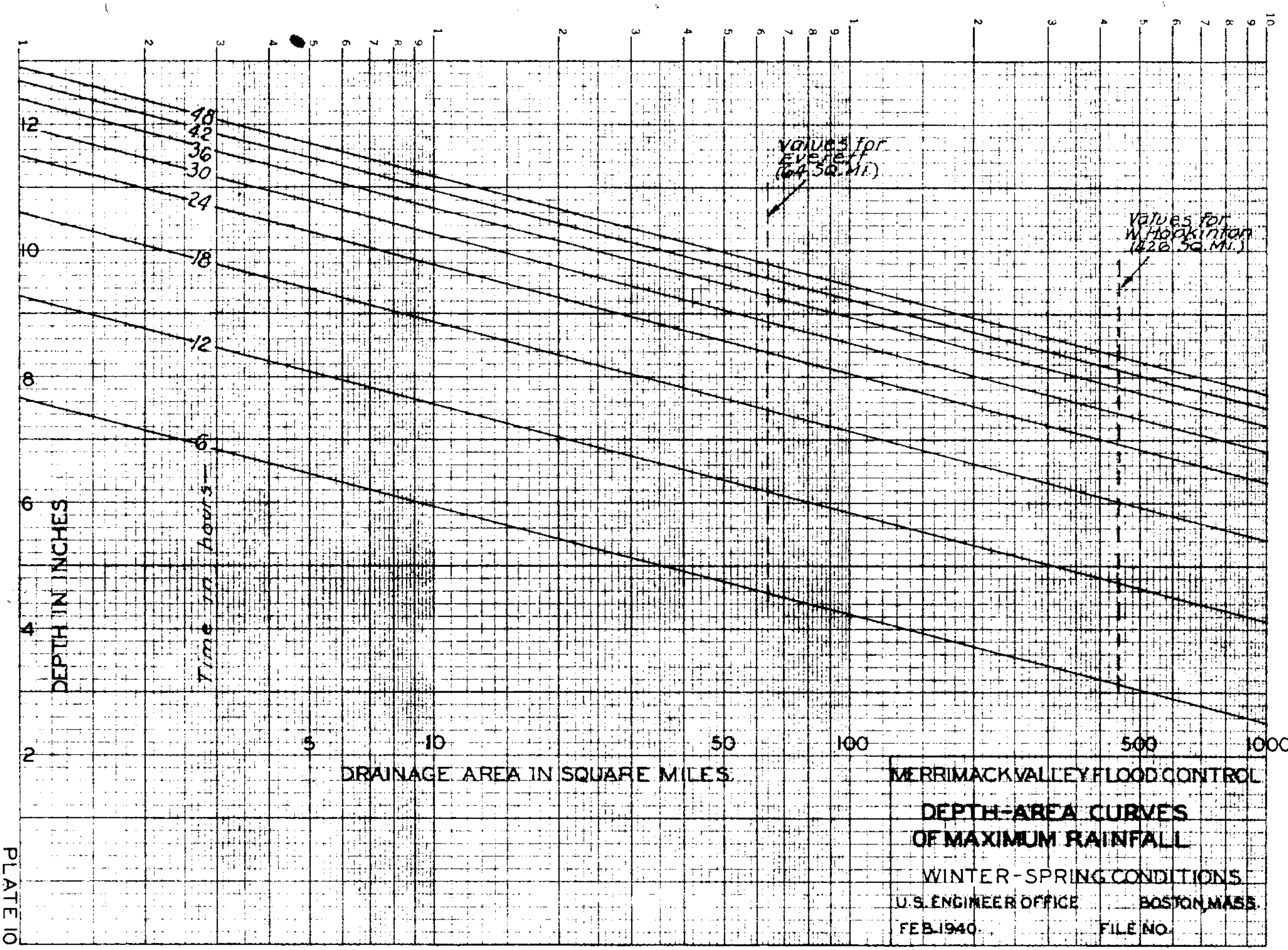


PLATE 9



DEPTH IN INCHES
Time in hours
48
42
36
30
24
18
12
6
Values for Everett (64 Sq. Mi.)
Values for W. Hopkinton (426 Sq. Mi.)
DRAINAGE AREA IN SQUARE MILES
MERRIMACK VALLEY FLOOD CONTROL
DEPTH-AREA CURVES OF MAXIMUM RAINFALL
WINTER-SPRING CONDITIONS
U.S. ENGINEER OFFICE
BOSTON, MASS.
FEB. 1940
FILE NO.

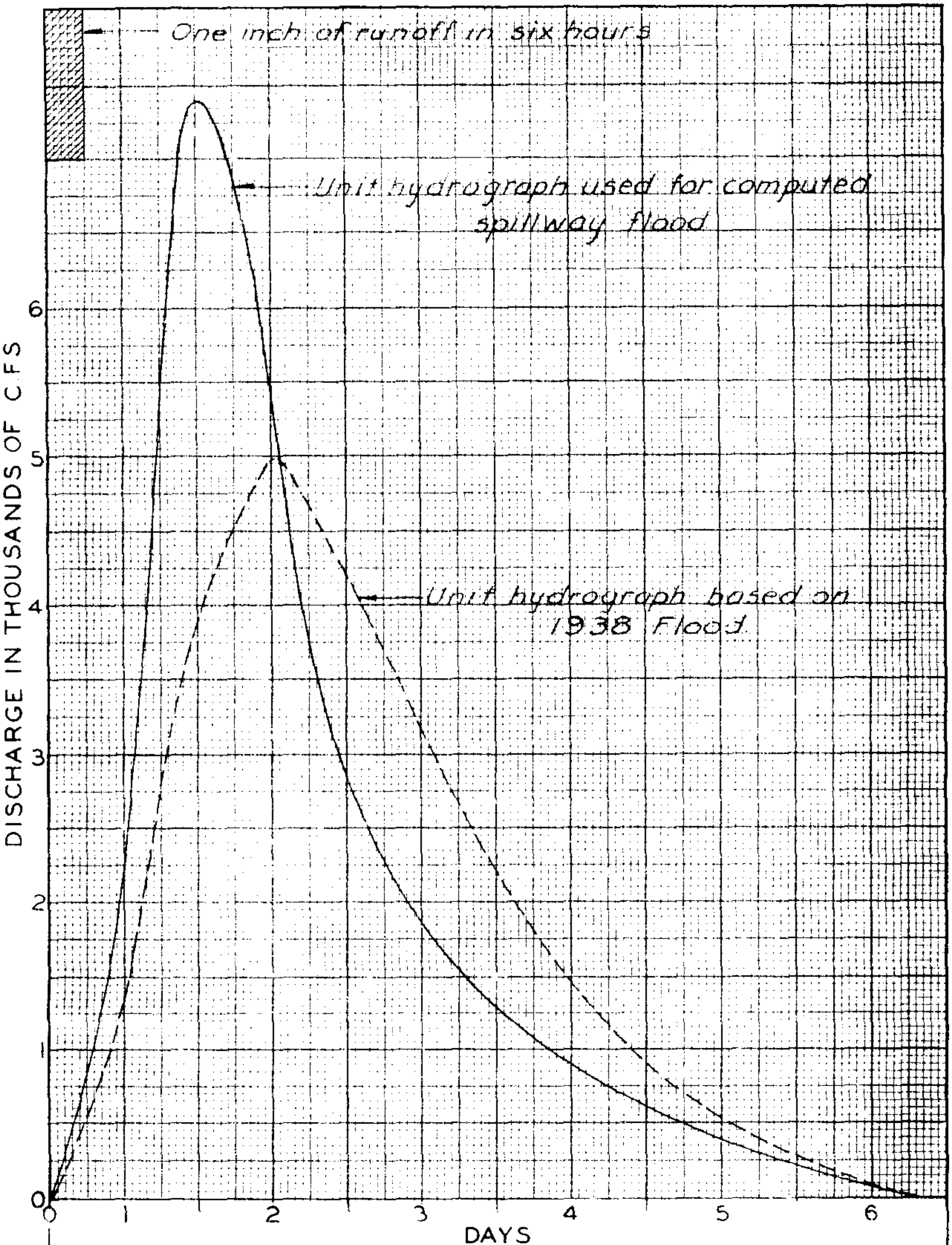
PLATE 10

HOPKINTON-EVERETT RESERVOIR.
W. HOPKINTON DAM
DERIVATION OF DISTRIBUTION VALUES

1 Day		2 Rainfall Runoff inches	3 Observed Hydrograph dsf 1/2 day	4 Base Flow dsf 1/2 day	5 Net Runoff dsf 1/2 day	6 Net Runoff dsf	7 1.0000	8 0.2425	9 0.0775	10 12 Hour Dist Values	11 6 Hour Dist Values
	A										
19	P	0.39	900	800	100	50			39		0.4
	A	1.22	1400	850	550	2[illegible]		123	398		1.8
20	P	0	2600	950	1650	825		1245	692		4.6
	A	5.03	8000	1100	6900	3450	508	2160	782	1.0	7.4
21	P		17700	1200	16500	8250	(4600) 5142	2440	668	9.4	9.0
	A		24400	1350	23050	11525	8925	2090	510	17.9	[illegible]
22	P		25500	1450	24050	12025	10092	1592	341	20.3	10.8
	A		21400	1600	19800	9900	8625	1068	207	17.3	9.2
23	P		16400	1700	14700	7350	6574	647	129	13.2	8.4
	A		11600	1850	9750	4875	4393	405	77	8.8	7.6
24	P		7900	2000	5900	2950	2671	240	39	5.4	6.6
	A		5700	2100	3600	1800	1667	121	12	3.4	5.6
25	P		4300	2250	2050	1025	989	36		2.0	4.4
	A		3400	2400	1000	500	500			1.0	3.9
26	P		2800	2500	300	150	150			0.3	2.8
											2.2
											1.6
											1.2
											1.0
											0.6
											0.6
											0.4
											0.2
		6.64	154,000	24100	129900	64950				100.0	100.0

Note: Column # 7 used for deriving distribution values in column # 10
6 hour values derived from 12 hour values

PLATE II

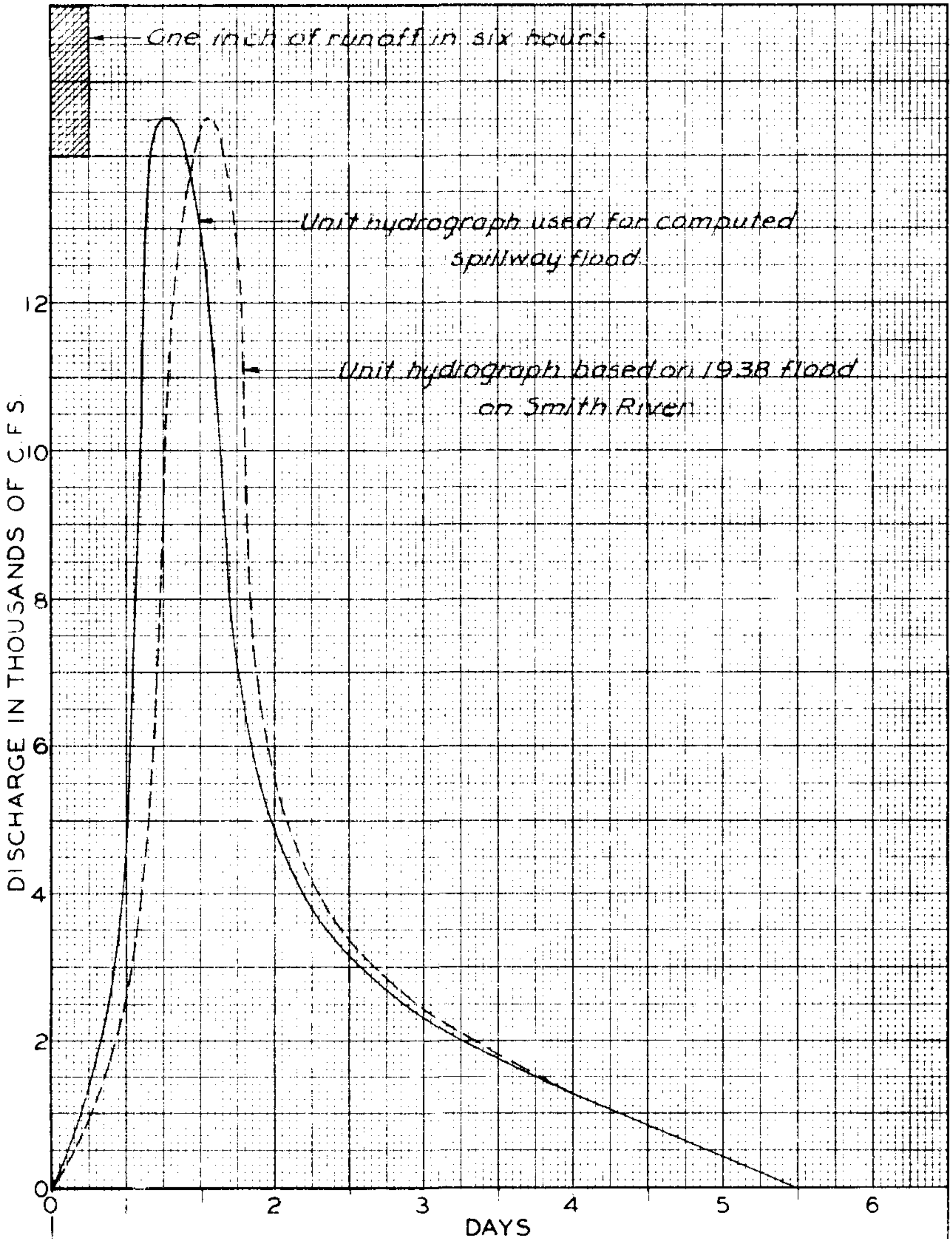


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
W. HOPKINTON DAM

INFLOW UNIT HYDROGRAPH

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 12



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
EVERETT DAM

INFLOW UNIT HYDROGRAPH

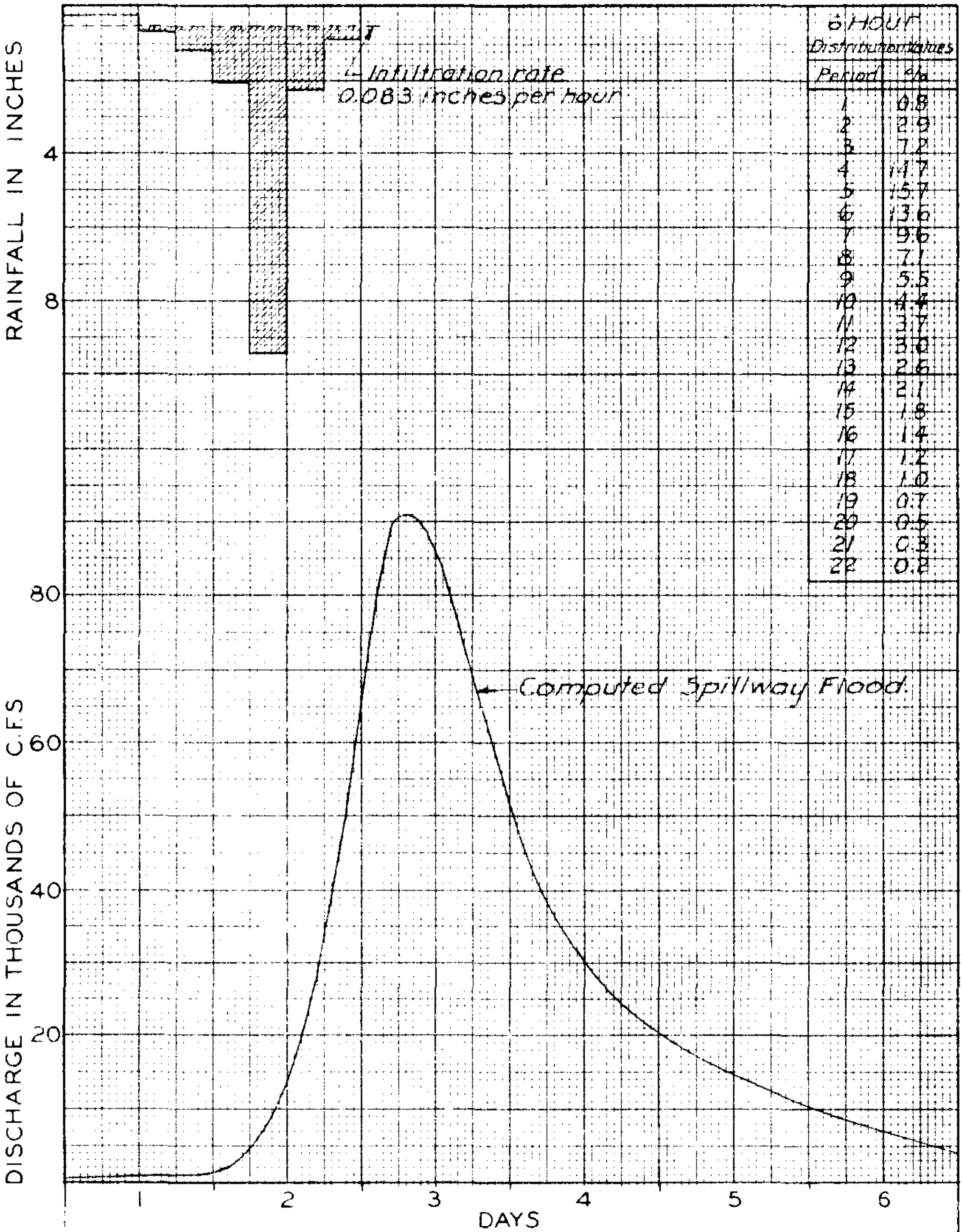
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 13

HOPKINTON - EVERETT RESERVOIR
— W. HOPKINTON DAM —
DERIVATION OF COMPUTED SPILLWAY FLOOD

Period		Rainfall inches	Run-off (Infilt rate 0.083"/hr) inches	Run-off dsf	Distribution Values %	Distributed Run-off — dsf 1	2	3	4	5	6	Total Run-off dsf	Total Run-off dsf 1/4 day	Total Run-off with base flow added dsf 1/4 day
	M	0.2	0	0										
	6A	0.2	0	0										
1	N	0.6	0.1	1146	0.8	9						9	36	886
	6P	1.1	0.6	6873	2.9	33	55					88	352	1202
	M	2.0	1.5	17183	7.2	83	199	137				419	1676	2526
	6A	9.4	8.9	101950	14.7	168	495	498	816			1977	7908	8758
2	N	2.2	1.7	19474	15.7	180	1010	1237	2957	156		5540	22160	23010
	6P	0.8	0.3	3437	13.6	156	1079	2526	7340	565	27	11693	46772	47622
	M				9.6	110	935	2698	14987	1402	100	20232	80928	81778
	6A				7.1	81	660	2337	16006	2863	248	22195	88780	89630
3	N				5.5	63	488	1650	13865	3057	505	19628	78512	79362
	6P				4.4	50	378	1220	9787	2648	540	14623	58492	59342
	M				3.7	42	302	945	7238	1870	468	10864	43456	44306
	6A				3.0	34	254	756	5607	1383	330	8364	33456	34306
4	N				2.6	30	206	636	4486	1071	244	6673	26692	27542
	6P				2.1	24	179	515	3772	857	189	5536	22144	22994
	M				1.8	21	144	447	3059	721	151	4543	18172	19022
	6A				1.4	16	124	361	2651	584	127	3863	15452	16302
5	N				1.2	14	96	309	2141	506	103	3169	12676	13526
	6P				1.0	11	82	241	1835	409	90	2667	10668	11518
	M				0.7	8	69	206	1427	351	72	2133	8532	9382
	6A				0.5	6	48	172	1223	273	62	1784	7136	7986
6	N				0.3	3	34	120	1020	234	48	1459	5836	6686
	6P				0.2	2	21	86	774	195	41	1059	4236	5086
	M						14	52	510	136	34	746	2984	3834
	6A							34	306	97	24	461	1844	2694
	N								204	58	17	279	1116	1966
	6P									39	10	49	196	1046
	M										7	7	28	878
		16.5	13.1	150063	100.0	1141	6872	17183	101951	19475	3435	150060	600240	

PLATE 14



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
W. HOPKINTON DAM

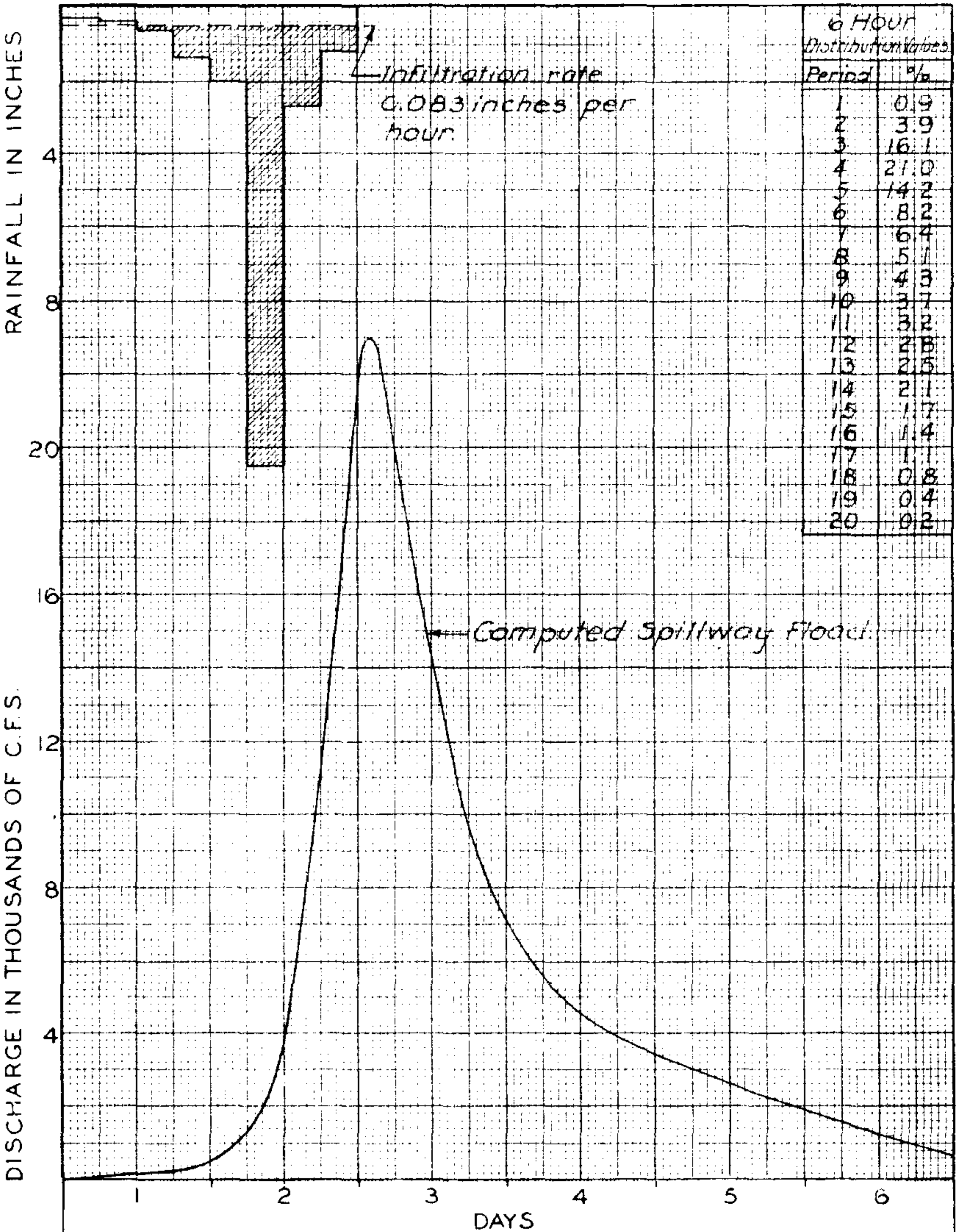
COMPUTED SPILLWAY FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

HOPKINTON-EVERETT RESERVOIR
— EVERETT DAM —
DERIVATION OF COMPUTED SPILLWAY FLOOD

Period		Rainfall inches	Run-off (infil. rate 0.083"/hr) inches	Run-off d.s.f.	Distribution Values %	Distributed Runoff d.s.f. 1	2	3	4	5	6	Total Runoff d.s.f.	Total Runoff dsf ½ day	Total Runoff with base flow added dsf ½ day
	M	0.3	0	0										
	6A	0.4	0	0										
1	N	0.6	0.1	172	0.9	2						2	8	133
	6P	1.3	0.8	1377	3.9	7	12					19	76	201
	M	2.0	1.5	2582	16.1	28	54	23				105	420	545
	6A	12.5	12.0	20652	21.0	36	222	101	186			545	2180	2305
2	N	2.7	2.2	3786	14.2	24	289	416	805	34		1568	6272	6397
	6P	1.2	0.7	1205	8.2	14	196	542	3325	148	11	4236	16944	17069
	M				6.4	11	113	367	4337	610	47	5485	21940	22065
	6A				5.1	9	88	212	2933	795	194	4231	16924	17049
3	N				4.3	7	70	165	1693	538	253	2726	10904	11029
	6P				3.7	6	59	132	1322	310	171	2000	8000	8125
	M				3.2	6	51	111	1053	242	99	1562	6248	6373
	6A				2.8	5	44	96	888	193	77	1303	5212	5337
4	N				2.5	4	39	83	764	163	61	1114	4456	4581
	6P				2.1	4	34	72	661	140	52	963	3852	3977
	M				1.7	3	29	65	578	121	45	841	3364	3489
	6A				1.4	2	23	54	516	106	39	740	2960	3085
5	N				1.1	2	19	44	434	95	34	628	2512	2637
	6P				0.8	1	15	36	351	80	30	513	2052	2177
	M				0.4	1	11	29	289	64	25	418	1672	1797
	6A				0.2	-	6	21	227	53	20	327	1308	1433
6	N						3	10	165	42	17	237	948	1073
	6P							5	83	30	13	131	524	649
	M								41	15	10	66	264	389
	6A									8	5	13	52	177
	N										2	2	8	133
	6P													
	M													
		21.0	17.3	29774	100.0							29775	119100	

PLATE 16



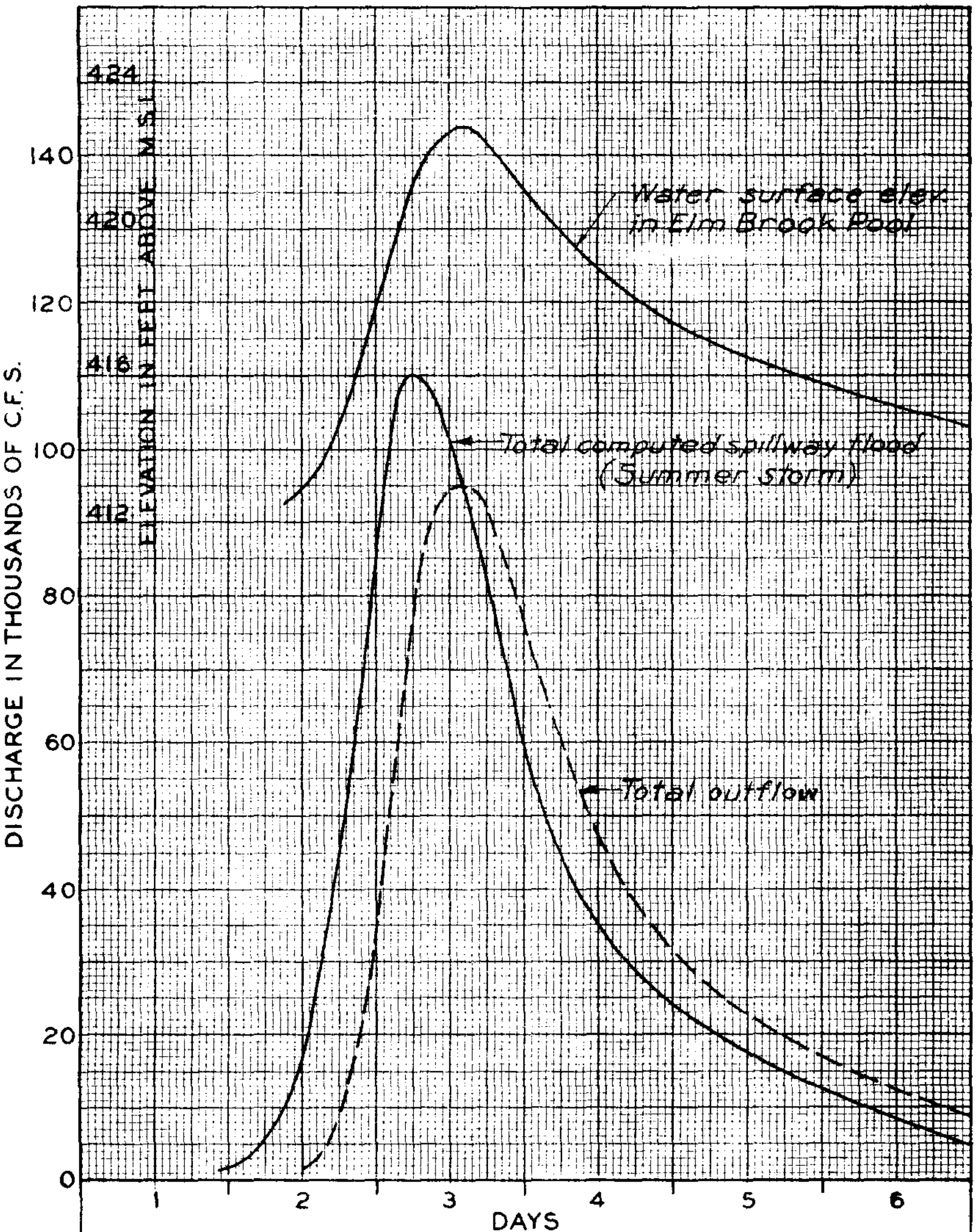
Period	%
1	0.9
2	3.9
3	16.1
4	21.0
5	14.2
6	8.2
7	6.4
8	5.1
9	4.3
10	3.7
11	3.2
12	2.8
13	2.5
14	2.1
15	1.7
16	1.4
17	1.1
18	0.8
19	0.4
20	0.2

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
EVERETT DAM

COMPUTED SPILLWAY FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 17



MERRIMACK VALLEY FLOOD CONTROL

HOPKINTON-EVERETT RESERVOIR

TOTAL COMPUTED SPILLWAY FLOOD

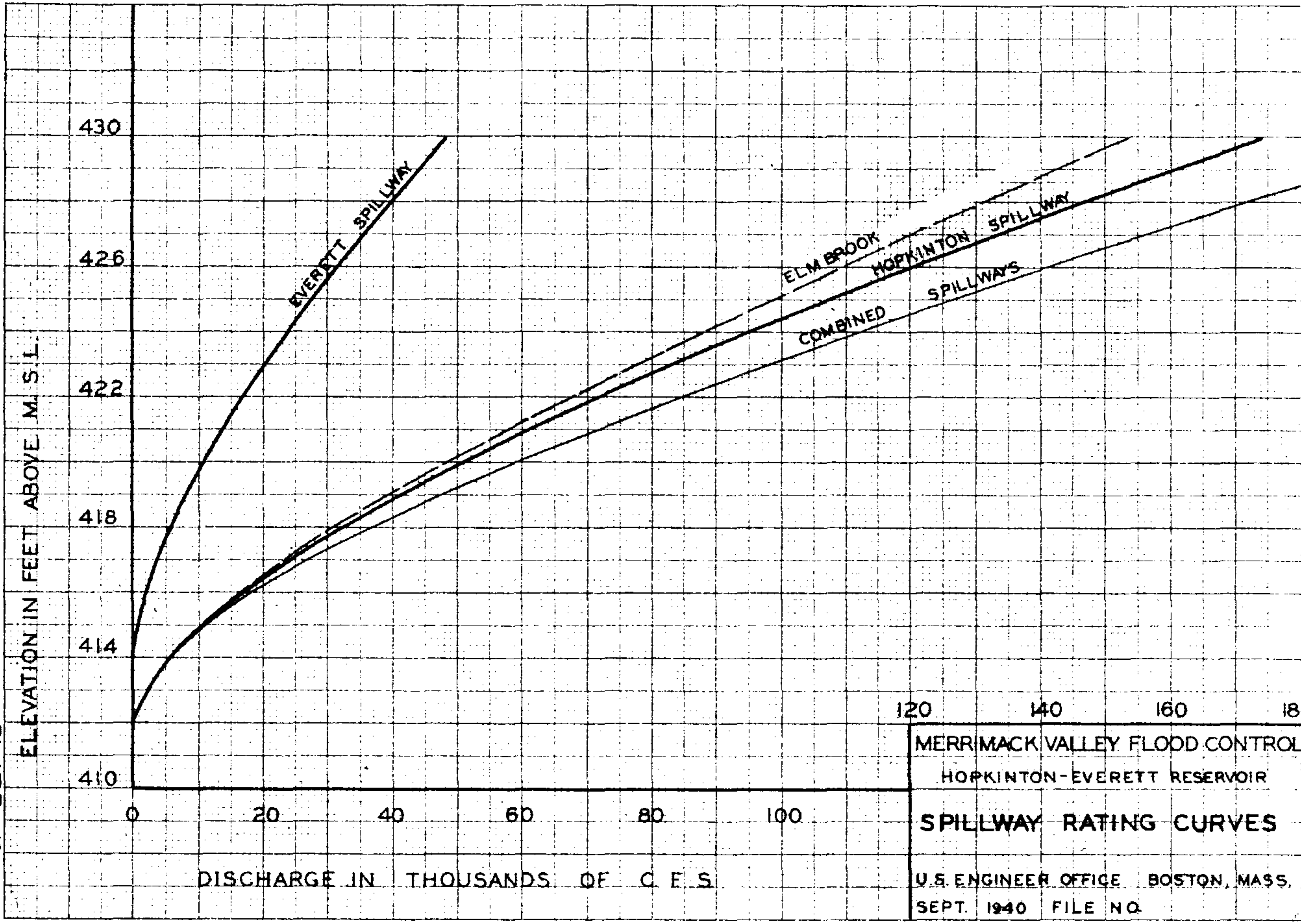
INFLOW-OUTFLOW-STAGE-HYDROGRAPH

U.S. ENGINEER OFFICE BOSTON, MASS.

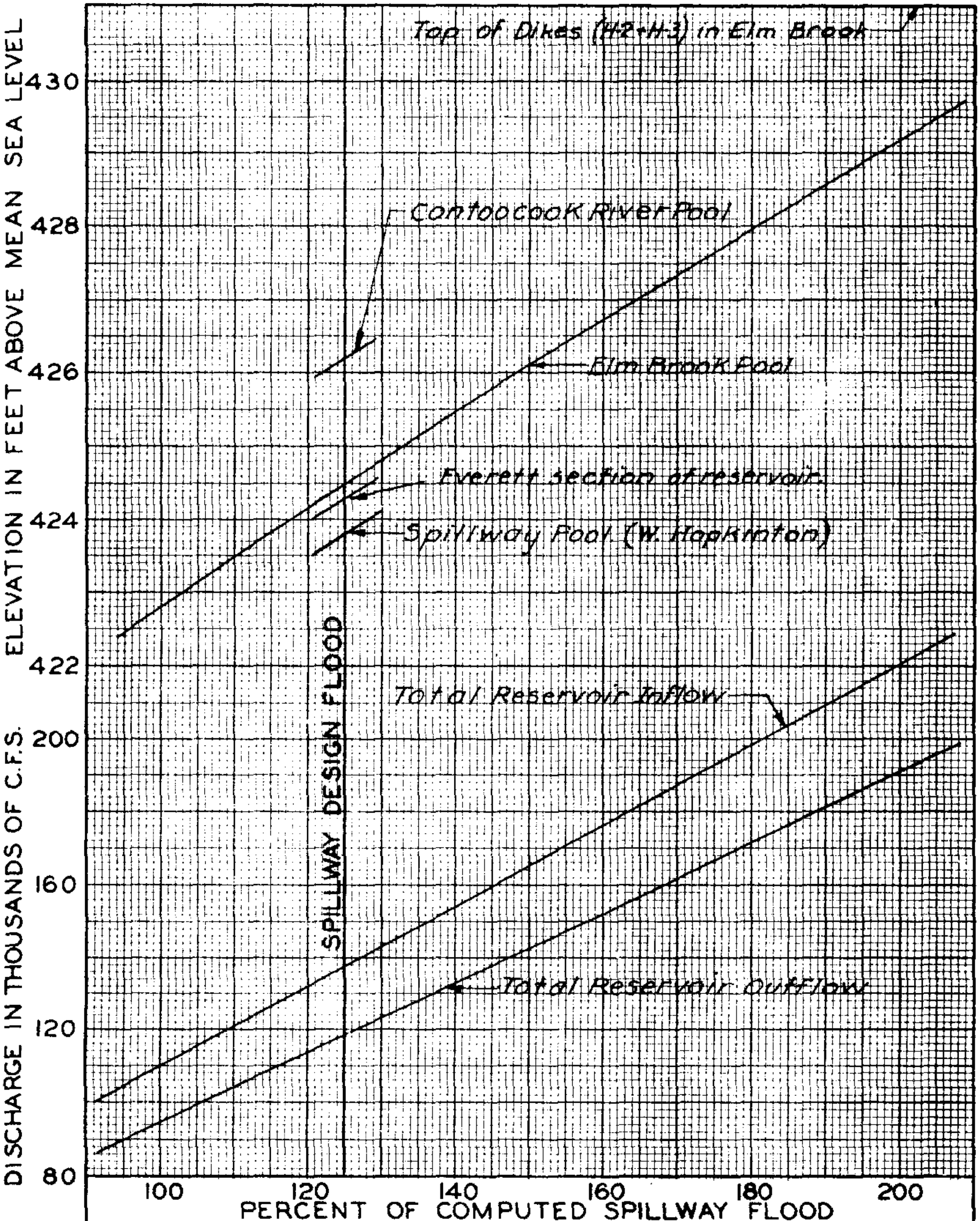
FEB. 1940 FILE NO.

PLATE 18

PLATE 19



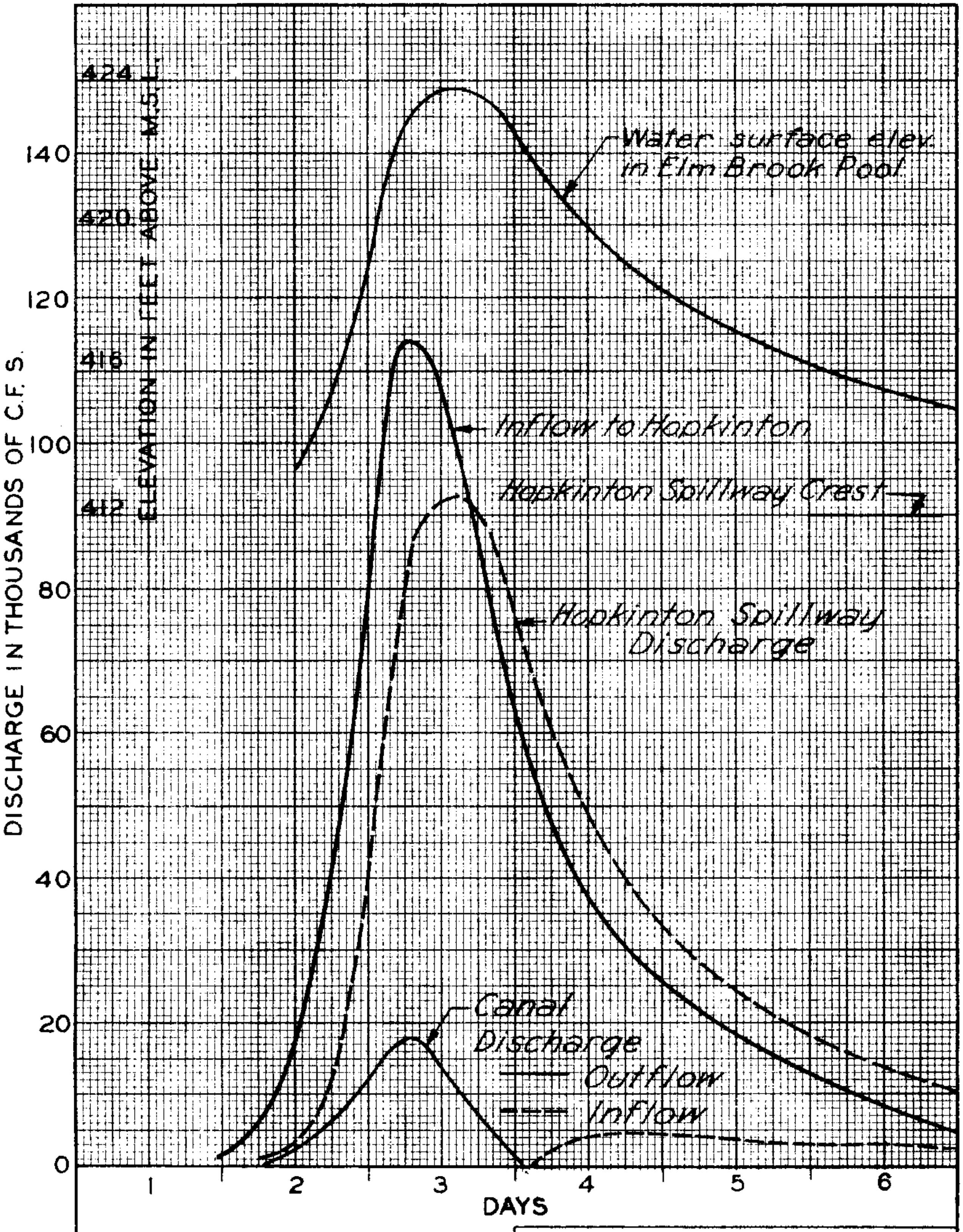
430
426
422
418
414
410
ELEVATION IN FEET ABOVE M. S. L.
0
20
40
60
80
100
120
140
160
180
DISCHARGE IN THOUSANDS OF C. F. S.
EVERETT SPILLWAY
ELM BROOK
HOPKINTON SPILLWAY
COMBINED SPILLWAYS
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
SPILLWAY RATING CURVES
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

PERCENT OF COMPUTED SPILLWAY FLOOD VS. PEAK FLOW AND POOL ELEVATION

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940. FILE NO.

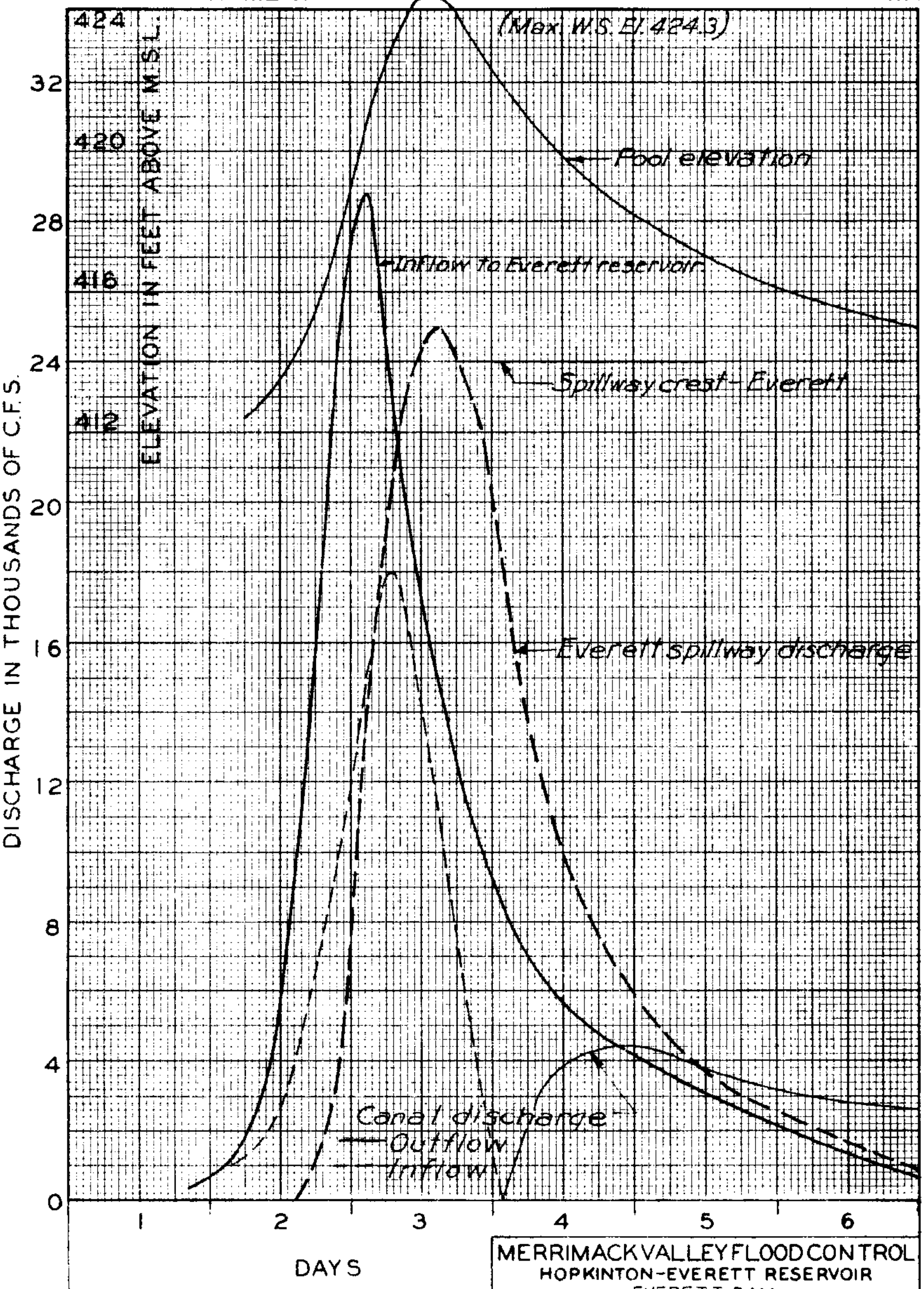


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
W. HOPKINTON DAM

SPILLWAY DESIGN FLOOD

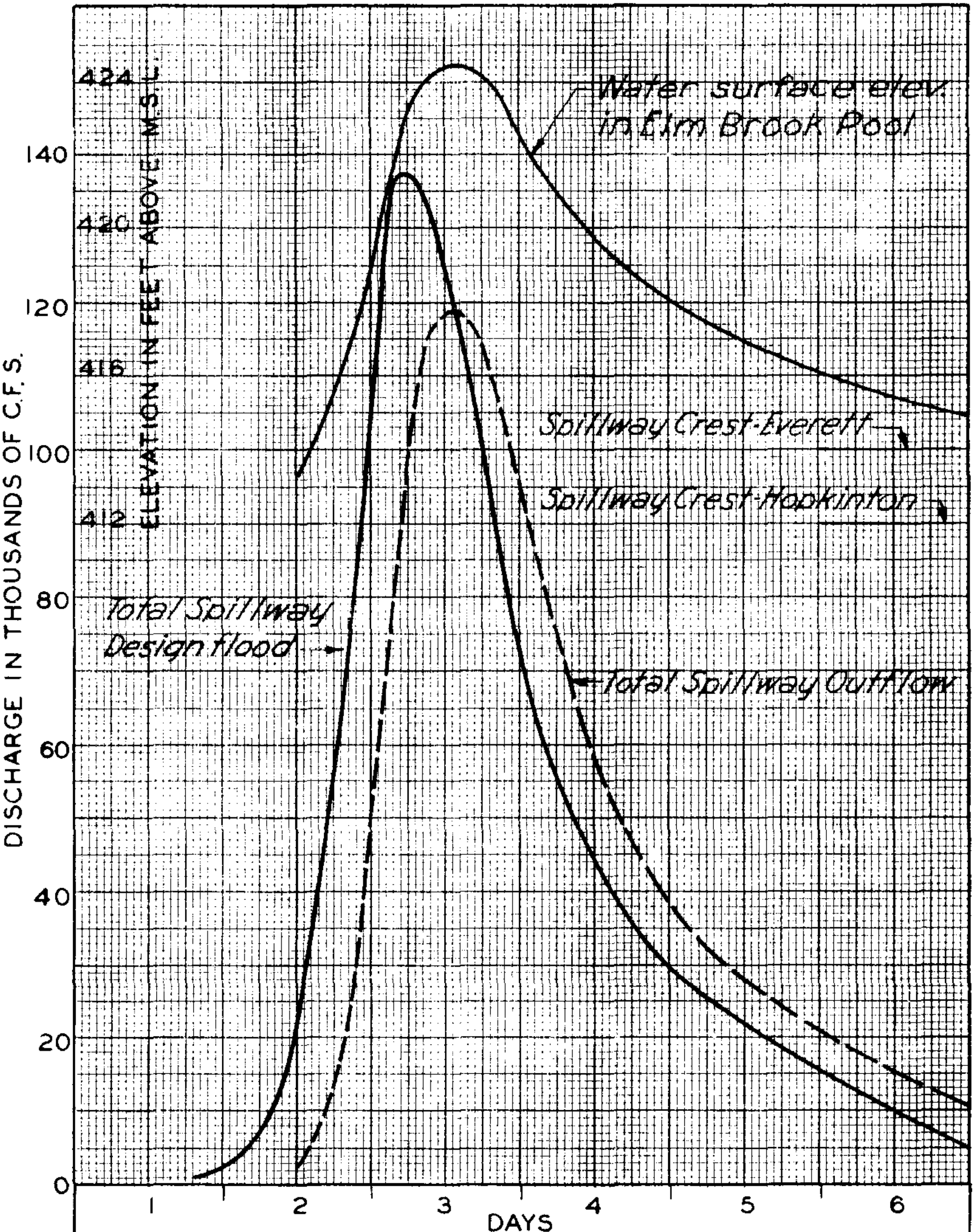
INFLOW-OUTFLOW-STAGE-HYDROGRAPH

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
EVERETT DAM
SPILLWAY DESIGN FLOOD
INFLOW-OUTFLOW-STAGE HYDROGRAPH
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 22



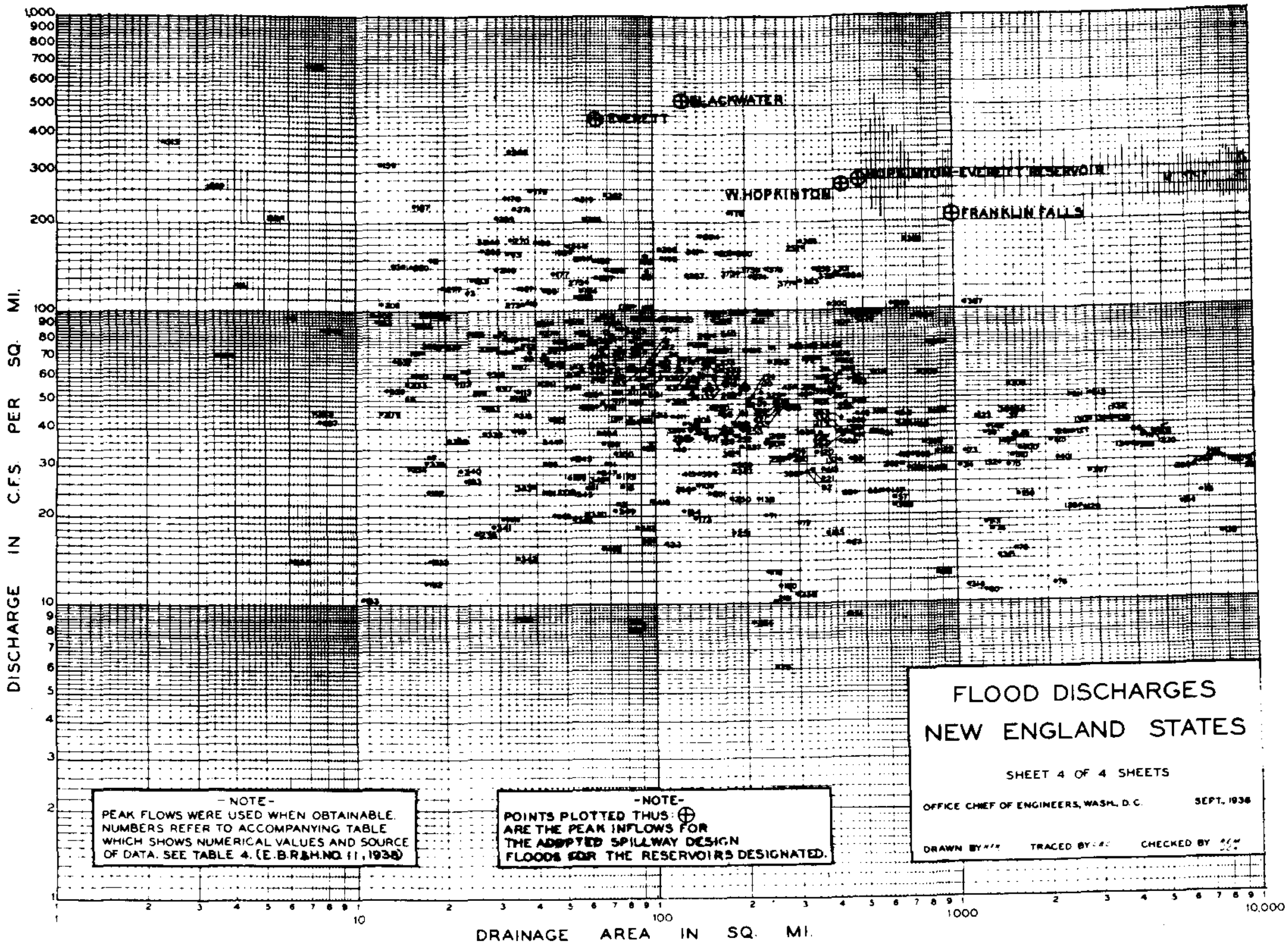
MERRIMACK VALLEY FLOOD CONTROL

HOPKINTON–EVERETT RESERVOIR

TOTAL SPILLWAY DESIGN FLOOD

INFLOW-OUTFLOW-STAGE HYDROGRAPH

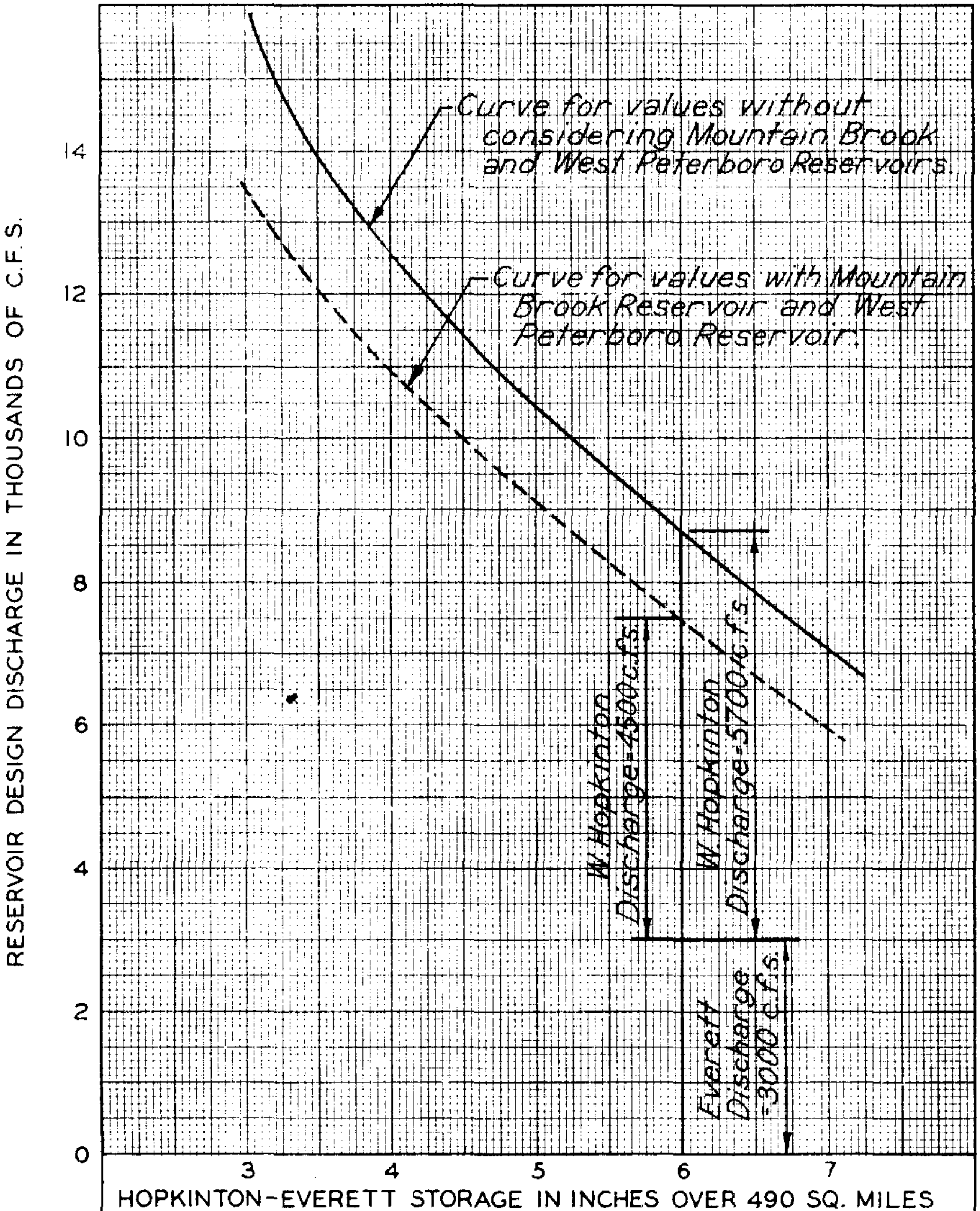
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.



FLOOD DISCHARGES
NEW ENGLAND STATES
SHEET 4 OF 4 SHEETS
OFFICE CHIEF OF ENGINEERS, WASH., D.C.
SEPT., 1938
DRAWN BY
TRACED BY
CHECKED BY
- NOTE -
PEAK FLOWS WERE USED WHEN OBTAINABLE. NUMBERS REFER TO ACCOMPANYING TABLE WHICH SHOWS NUMERICAL VALUES AND SOURCE OF DATA. SEE TABLE 4. (E.B.R.&H. NO. 11, 1938)
- NOTE -
POINTS PLOTTED THUS: ⊕ ARE THE PEAK INFLOWS FOR THE ADOPTED SPILLWAY DESIGN FLOODS FOR THE RESERVOIRS DESIGNATED.
BLACKWATER
EVERETT
W. HOPKINTON
HOPKINTON-EVERETT RESERVOIR
FRANKLIN FALLS
DISCHARGE IN C.F.S. PER SQ. MI.
DRAINAGE AREA IN SQ. MI.
1,000
100
10
1000
10,000

PLATE 24

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



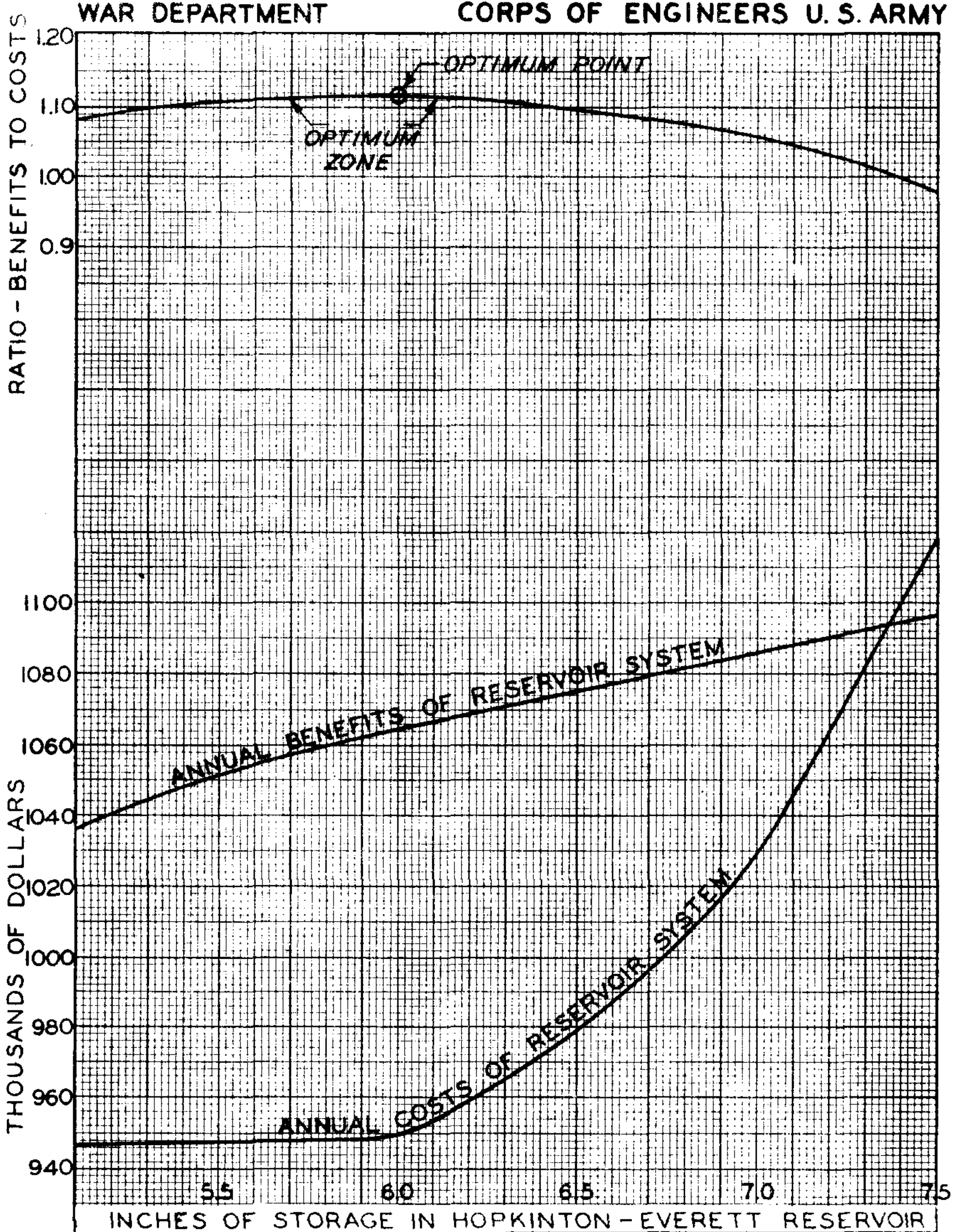
MERRIMACK VALLEY FLOOD CONTROL

HOPKINTON-EVERETT RESERVOIR

STORAGE VS DISCHARGE

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 25

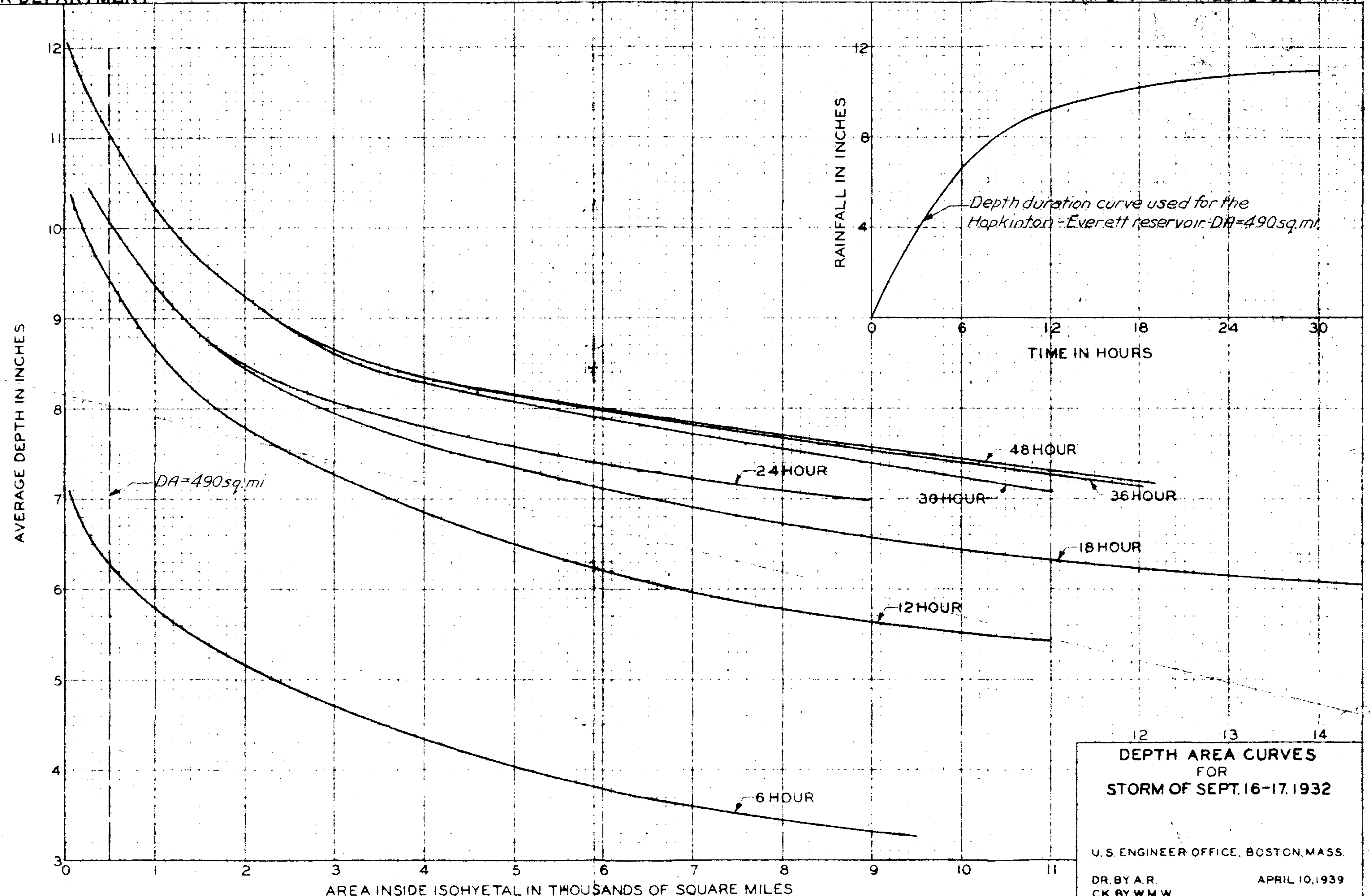


Note:
The annual benefits and costs shown are for the reservoir system-Franklin Falls, Blackwater and Hopkinton-Everett.

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

ECONOMIC STORAGE CAPACITY STUDY

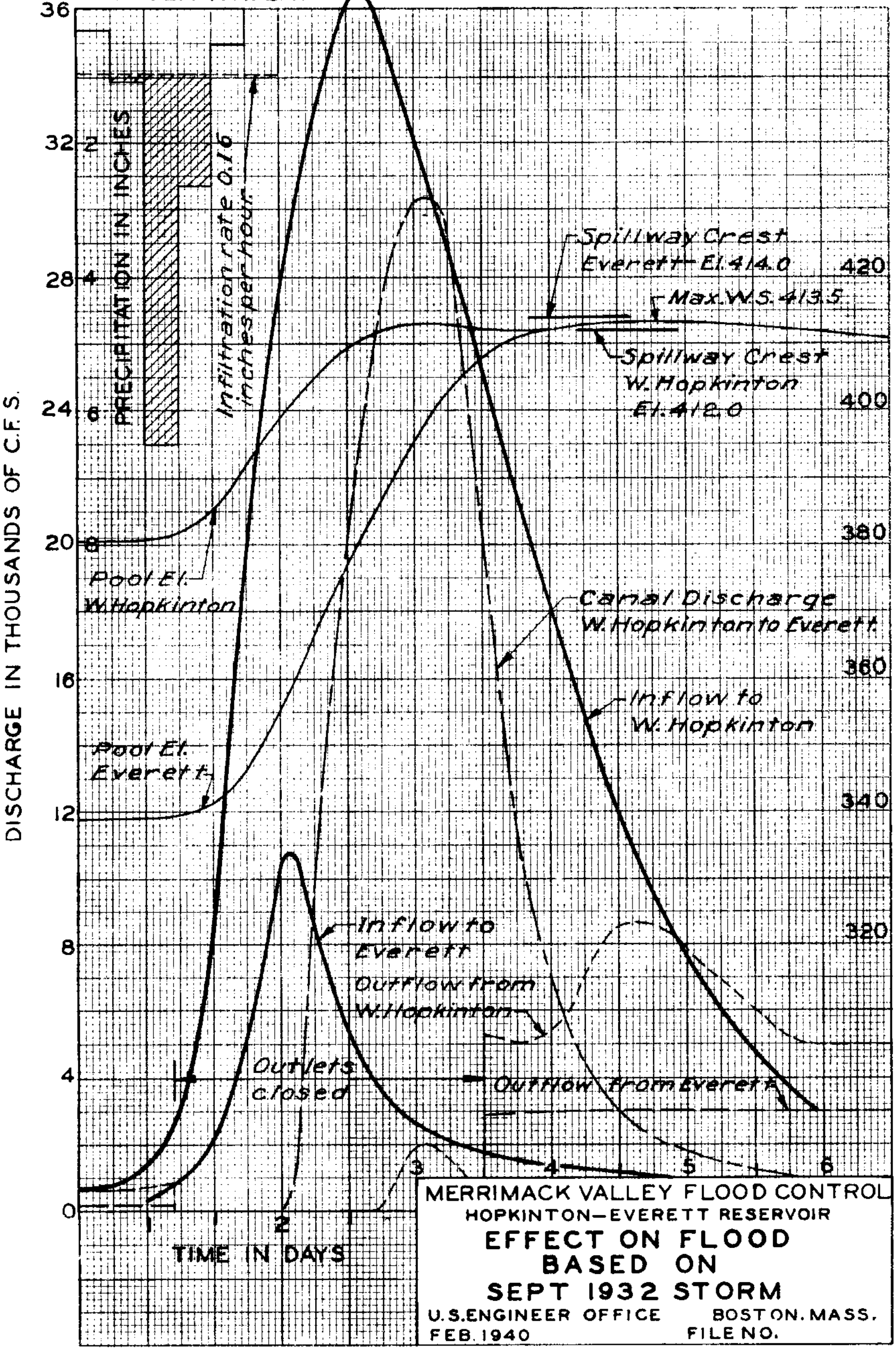
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.



DEPTH AREA CURVES
FOR
STORM OF SEPT. 16-17, 1932

U.S. ENGINEER OFFICE, BOSTON, MASS.

DR. BY A.R. APRIL 10, 1939
CK. BY W.M.W.

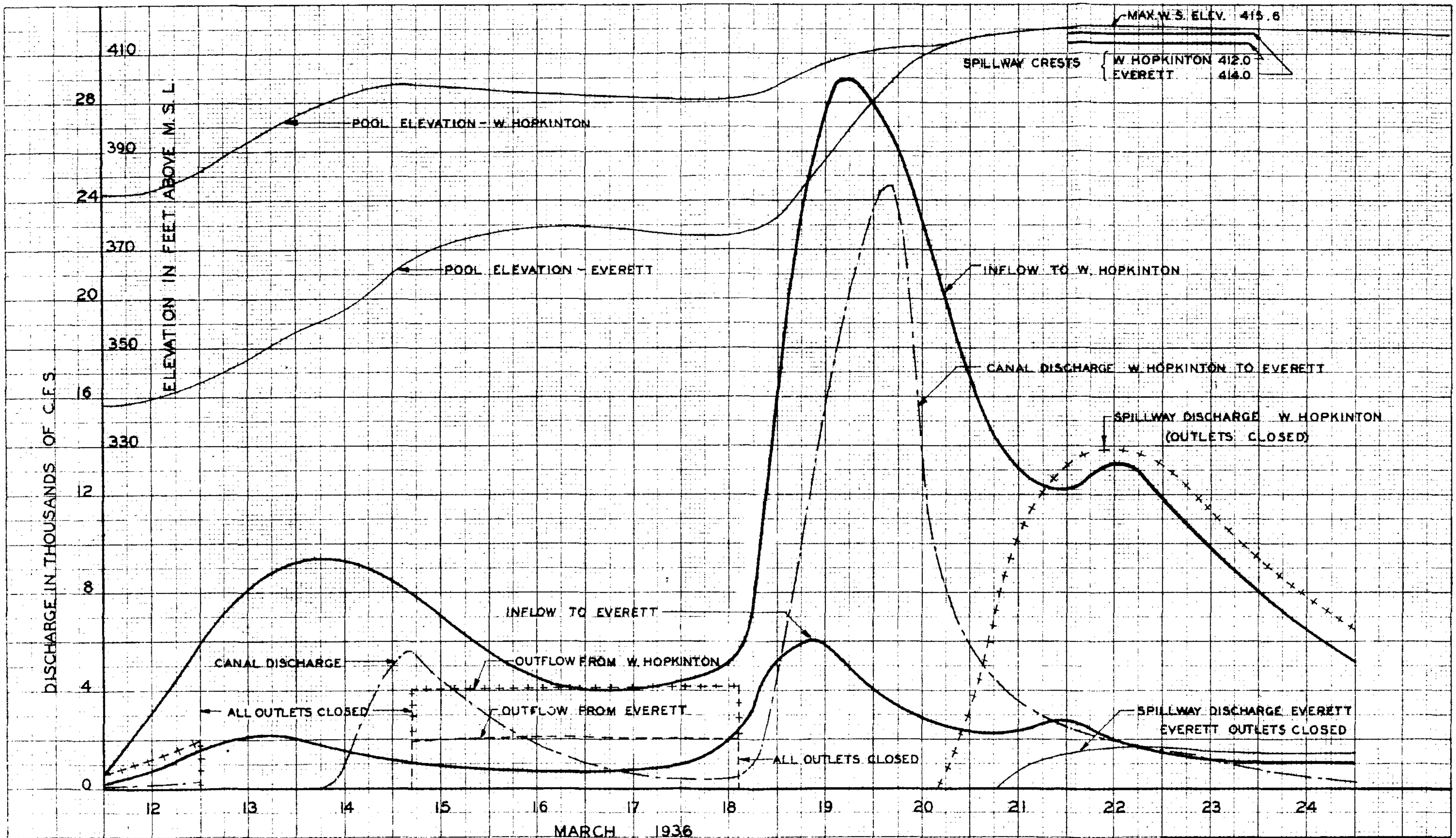


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON—EVERETT RESERVOIR

EFFECT ON FLOOD BASED ON SEPT 1932 STORM

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 28

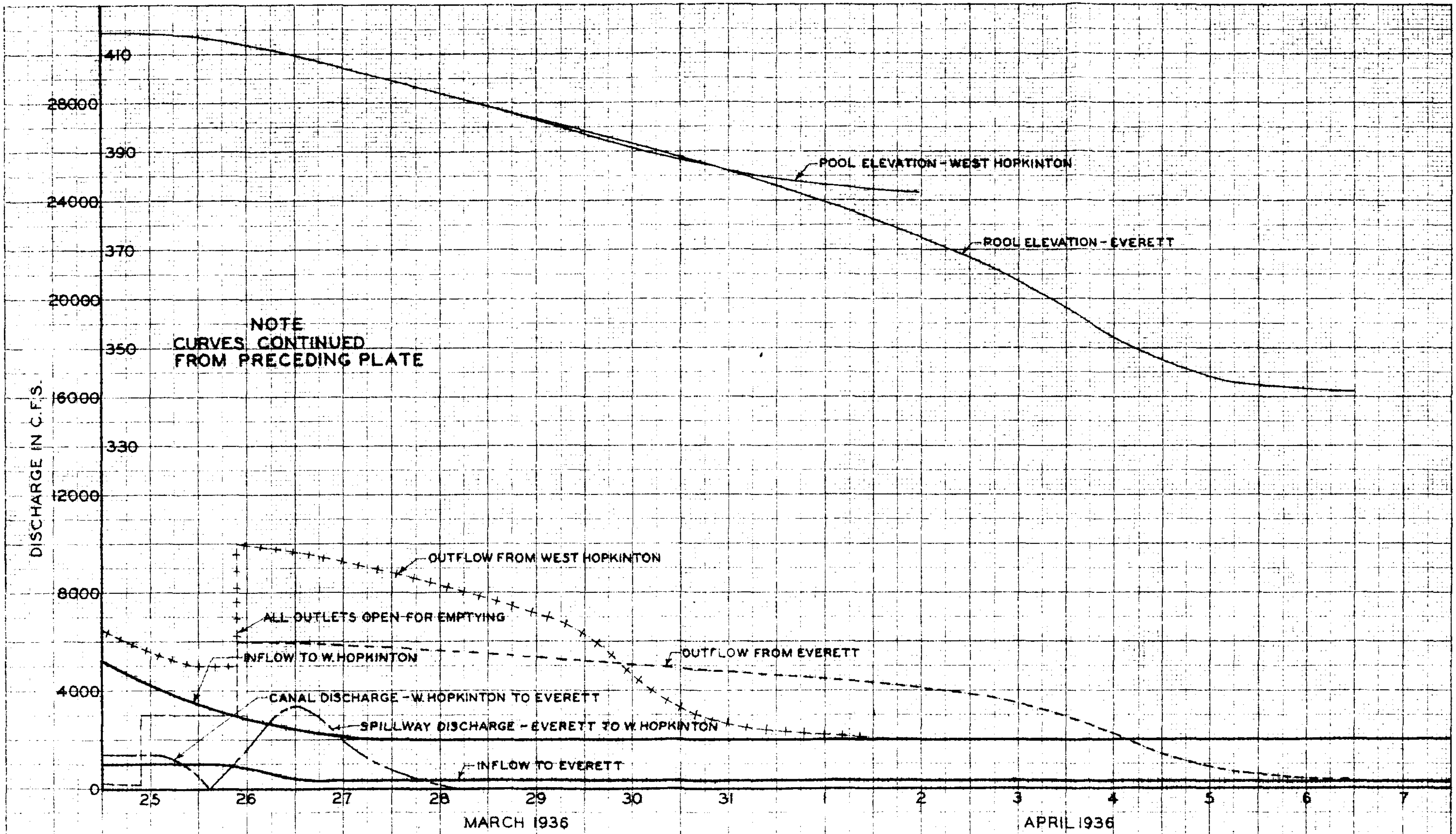


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

EFFECT ON 1936 FLOOD

U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPT 1940 FILE NO.

PLATE 29A

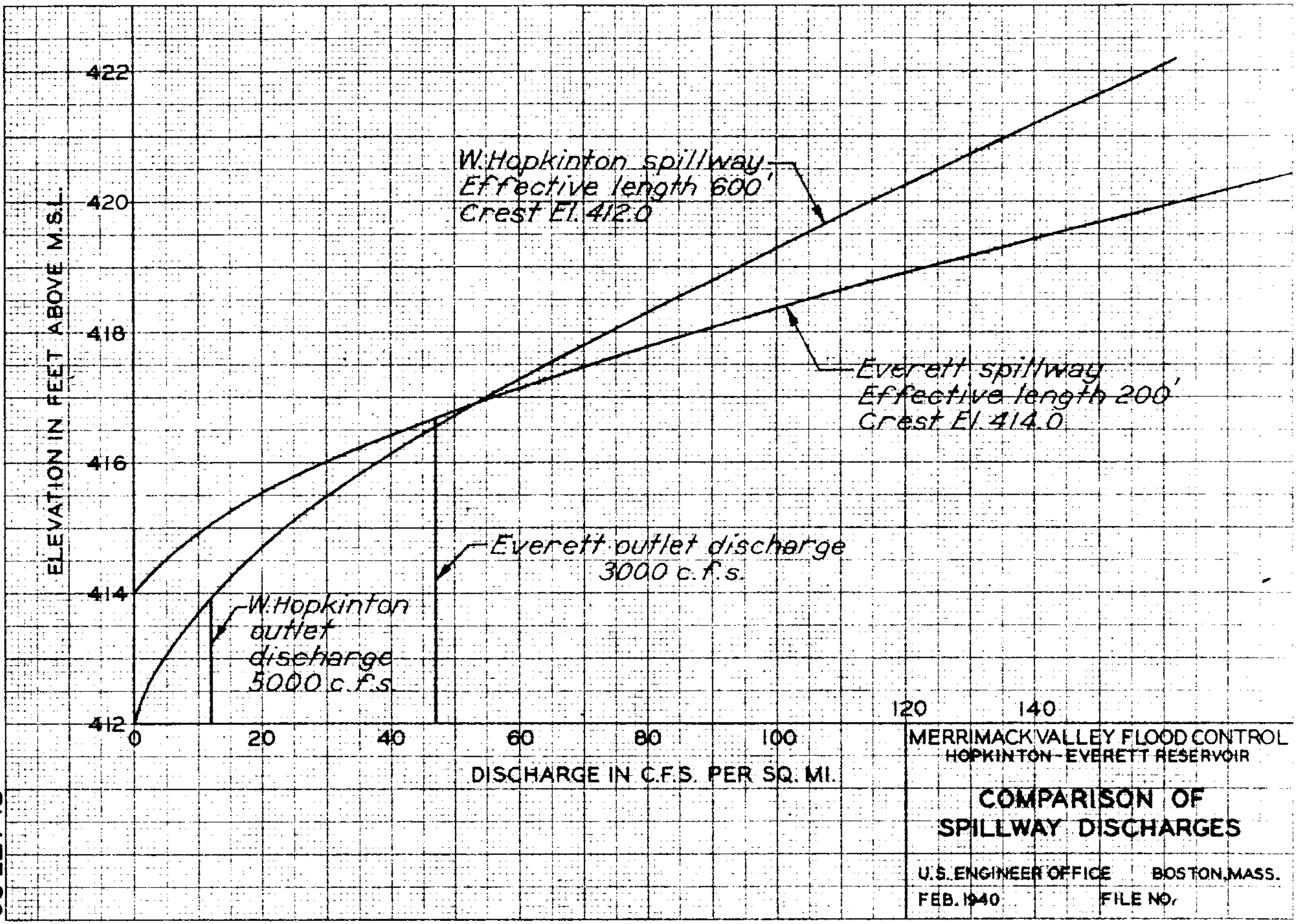


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR

EFFECT ON 1936 FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 29B

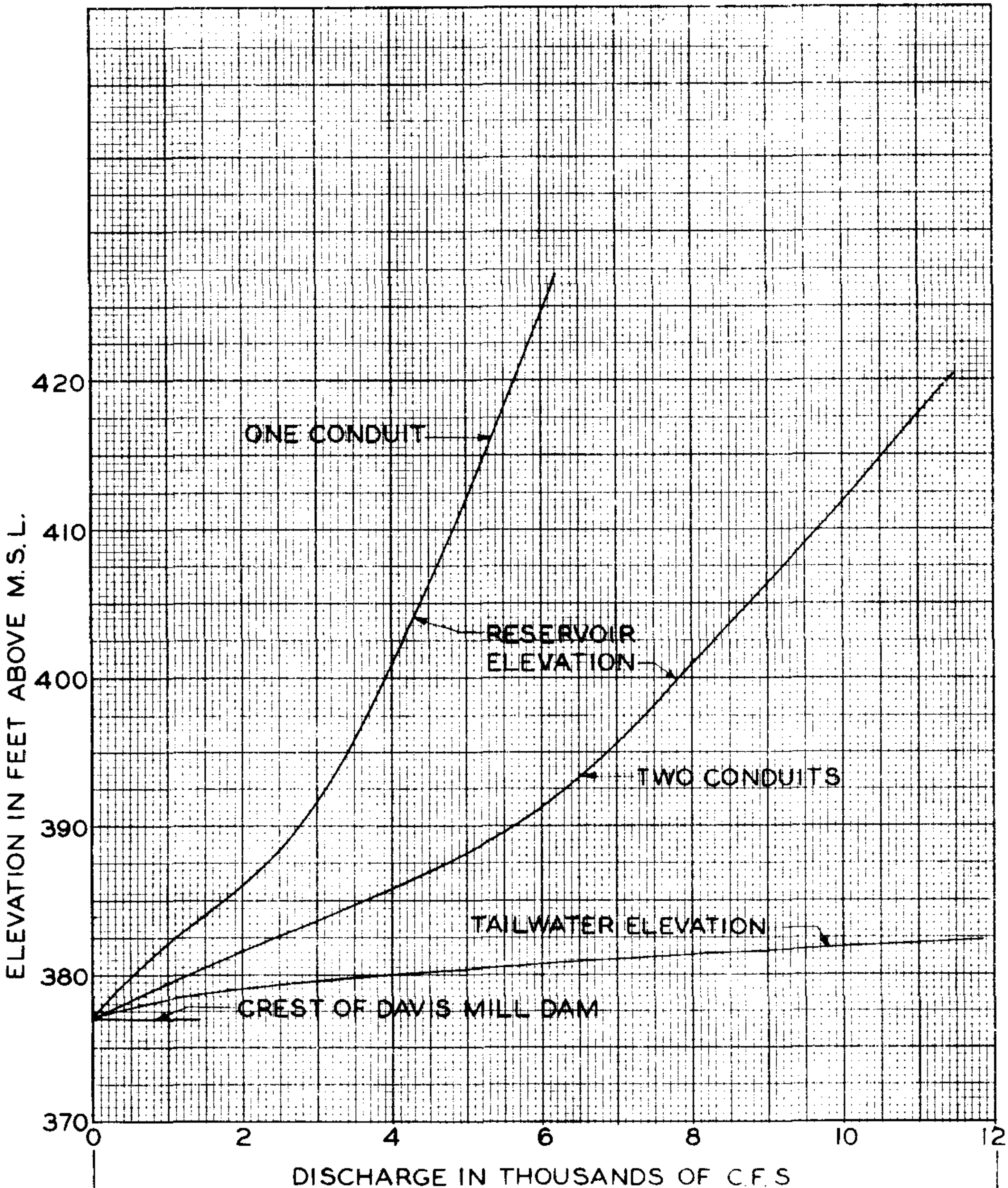


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR

COMPARISON OF SPILLWAY DISCHARGES

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 30

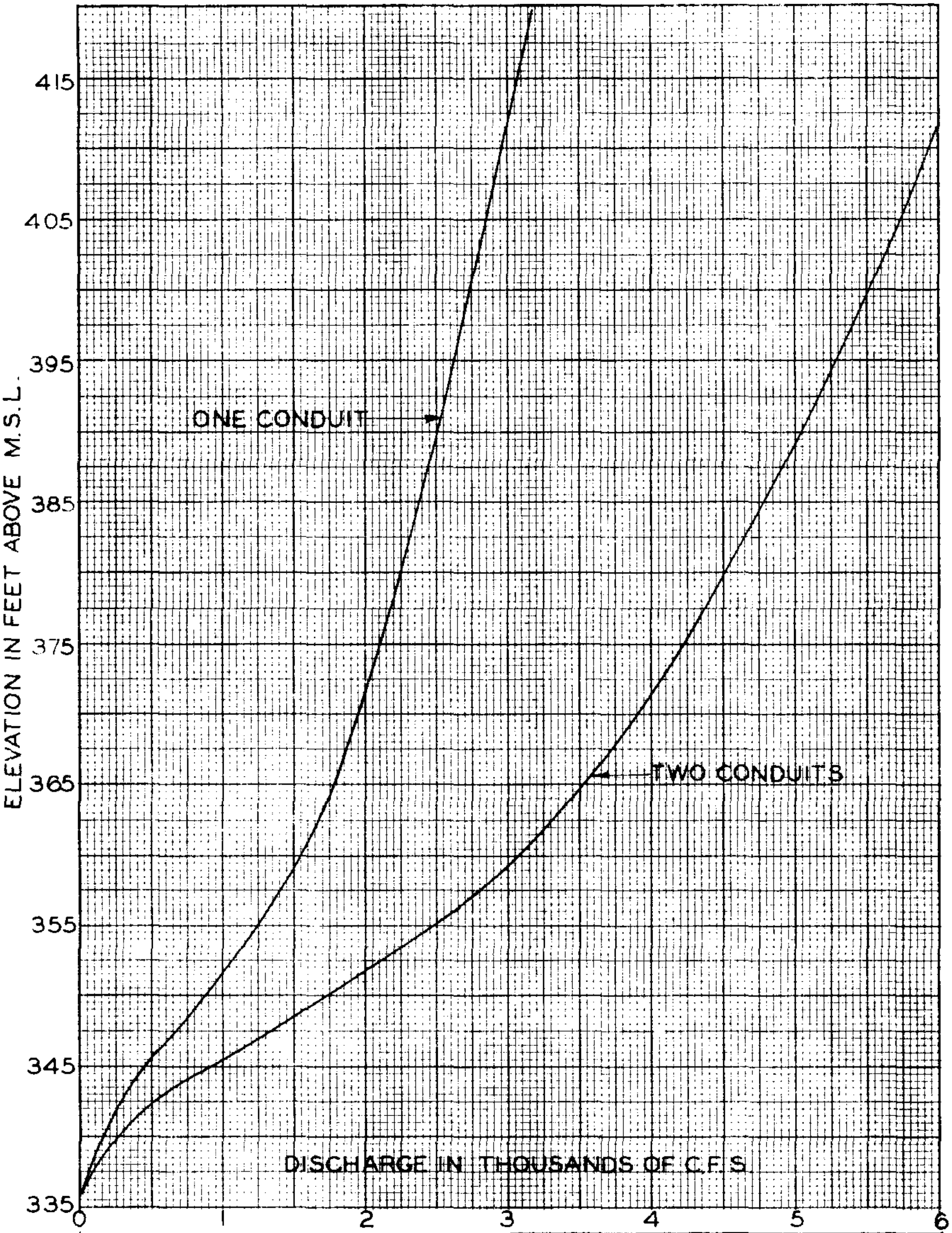


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
W. HOPKINTON DAM

OUTLET AND TAILWATER RATING CURVES

U.S. ENGINEER OFFICE BOSTON, MASS.

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
EVERETT DAM

OUTLET RATING CURVE

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 32

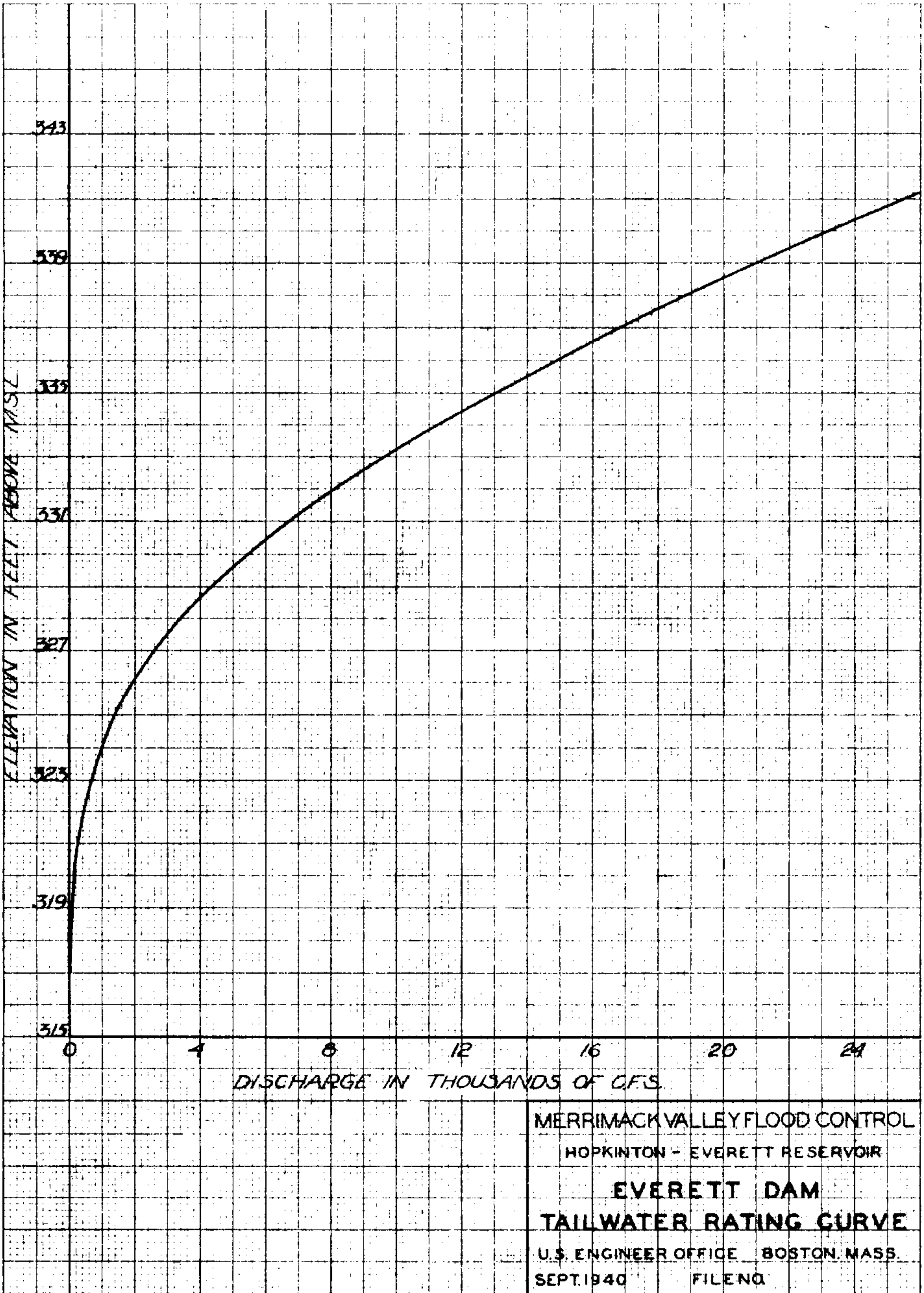
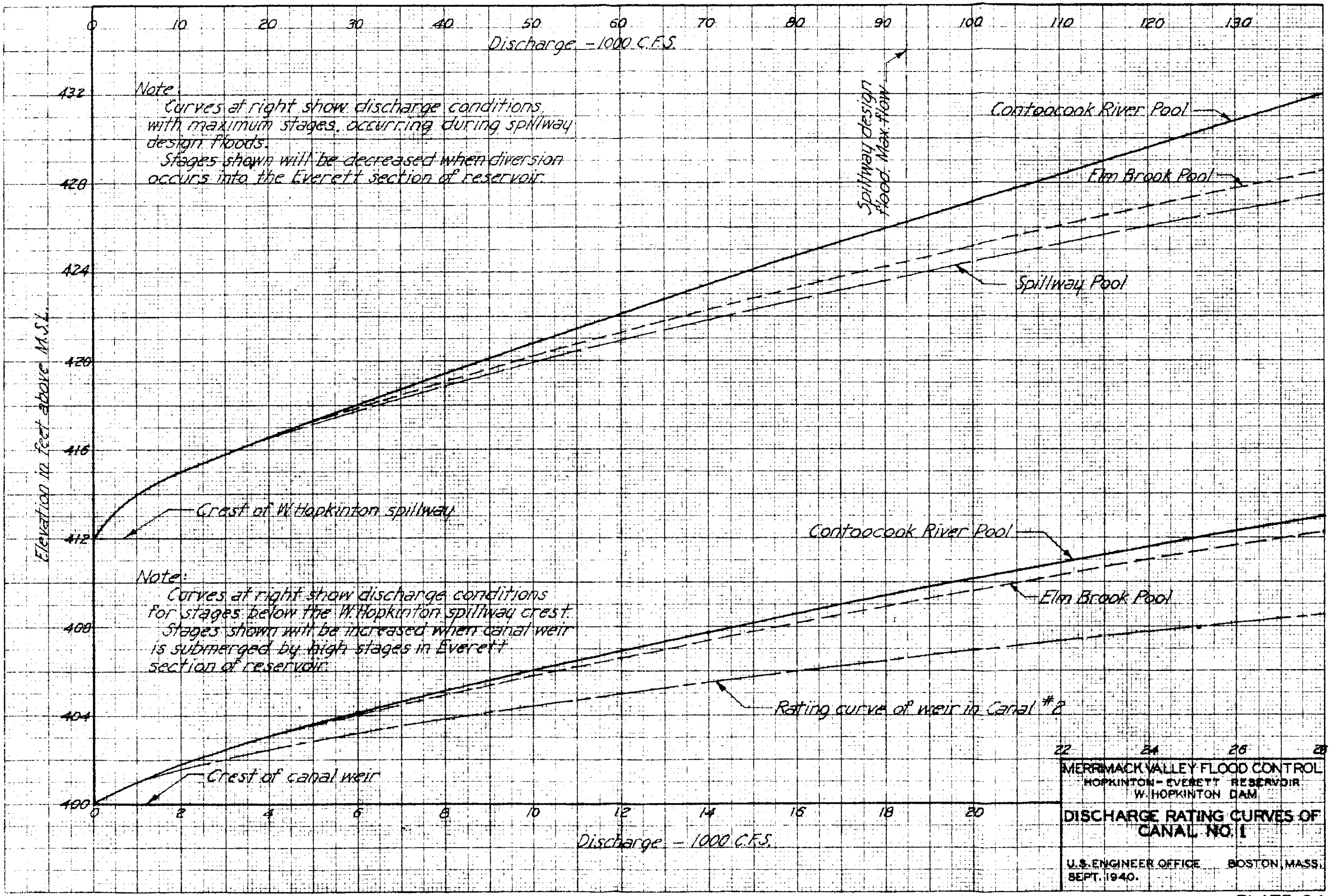

343
339
335
331
327
323
319
315
ELEVATION IN FEET ABOVE MSL
0
4
8
12
16
20
24
DISCHARGE IN THOUSANDS OF C.F.S.
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
EVERETT DAM
TAILWATER RATING CURVE
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 33

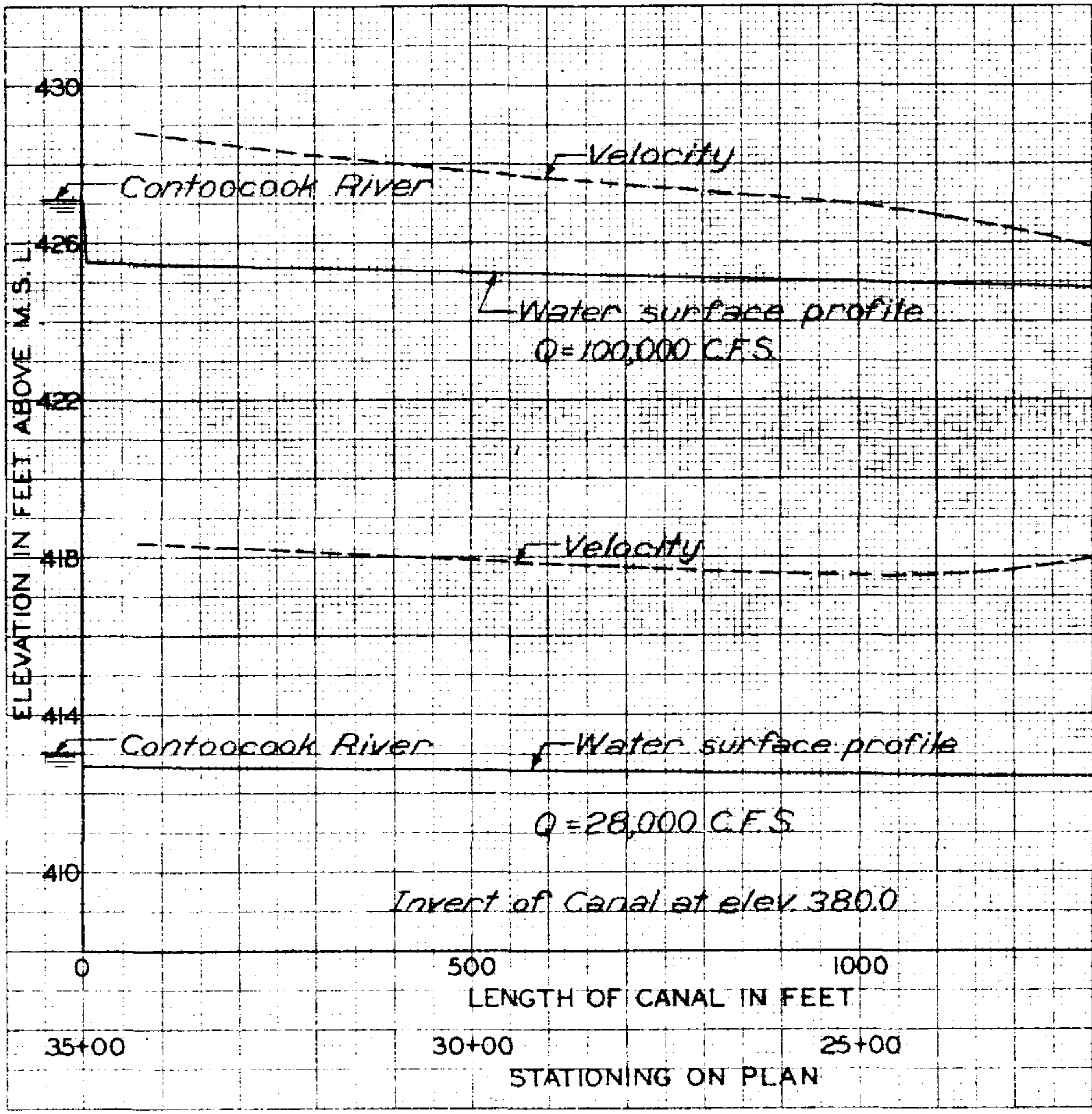


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
W. HOPKINTON DAM

DISCHARGE RATING CURVES OF CANAL NO. 1

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940.

PLATE 34



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
WATER SURFACE PROFILES
AND VELOCITIES
CANAL NO. 1
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 35

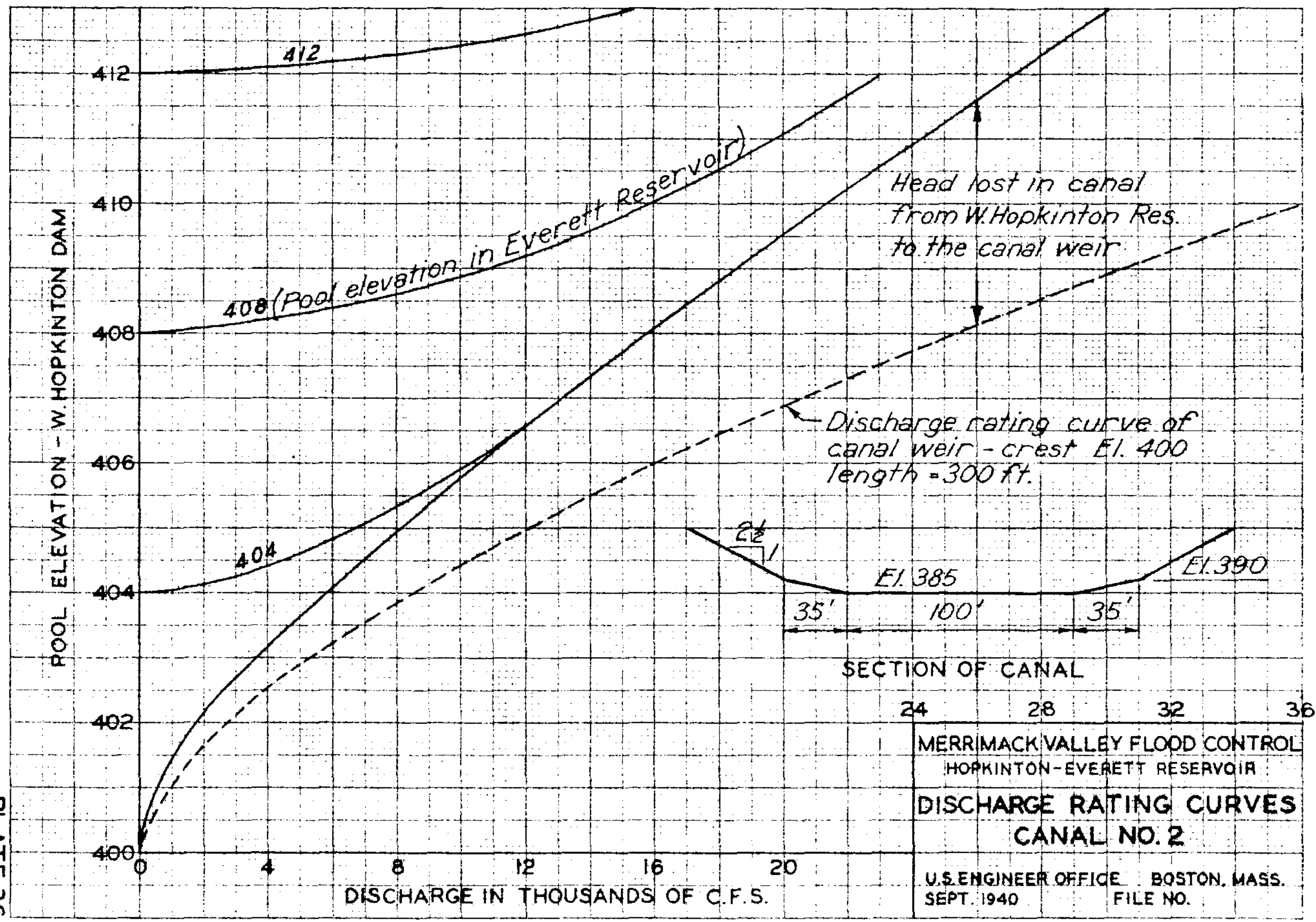
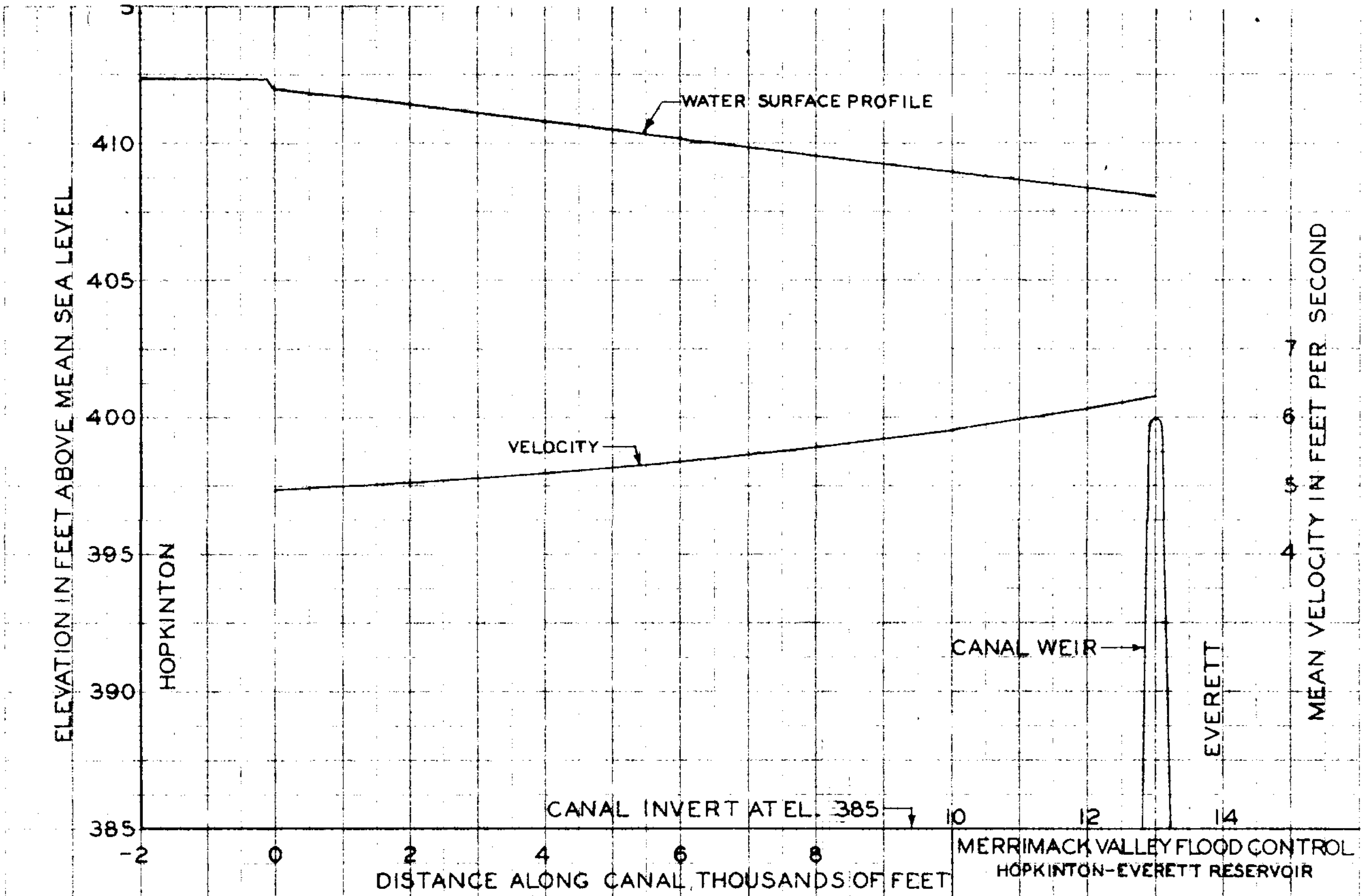

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
DISCHARGE RATING CURVES
CANAL NO. 2
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940
FILE NO.
POOL ELEVATION - W. HOPKINTON DAM
DISCHARGE IN THOUSANDS OF C.F.S.
412
408 (Pool elevation in Everett Reservoir)
404
Head lost in canal from W. Hopkinton Res. to the canal weir
Discharge rating curve of canal weir - crest El. 400 Length = 300 ft.
El. 385
El. 390
35'
100'
35'
SECTION OF CANAL

PLATE 36



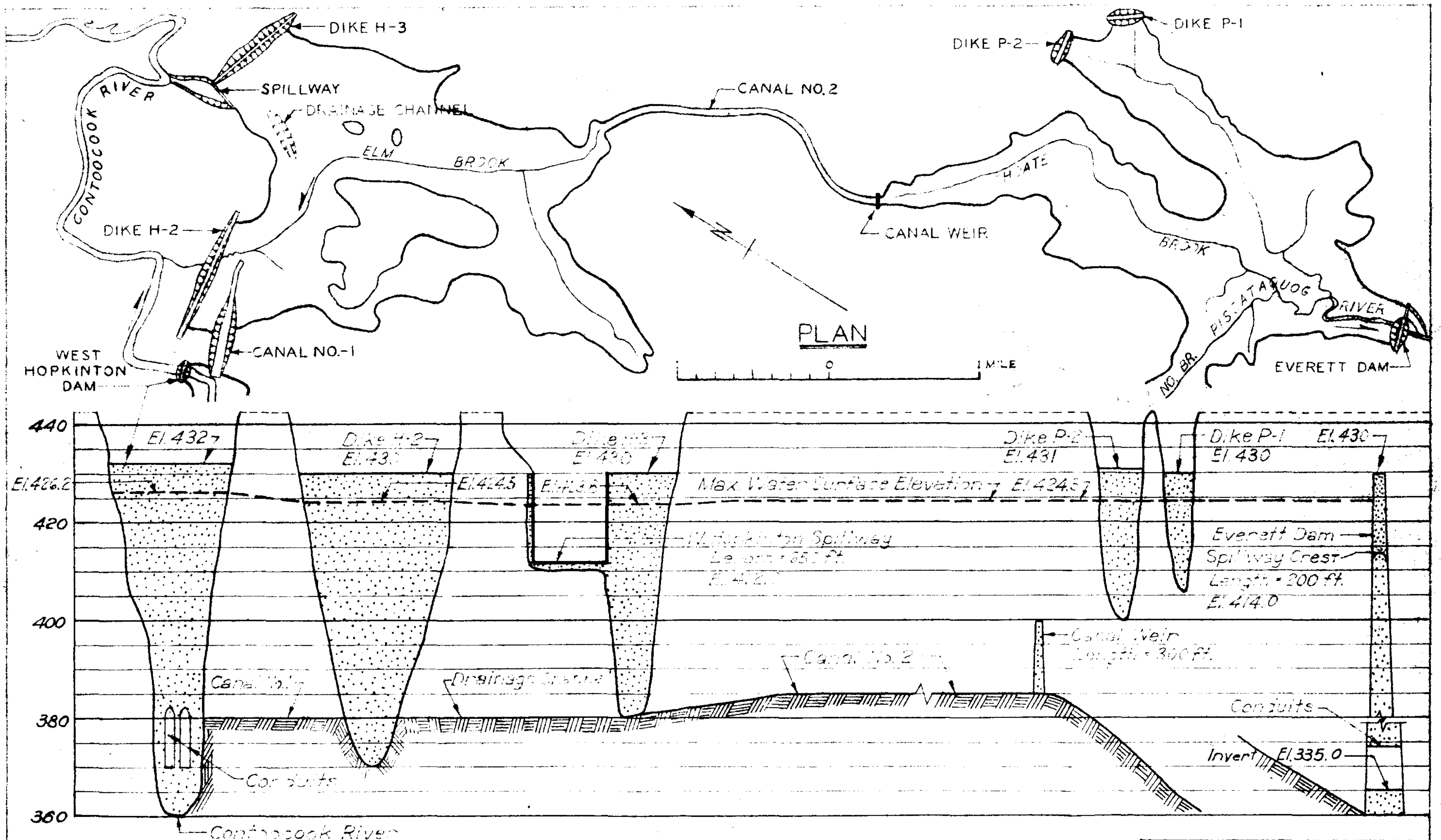
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

WATER SURFACE PROFILE AND VELOCITY-CANAL NO. 2.

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940. FILE NO.

NOTE: - DISCHARGE = 28000 C.F.S. POOL ELEVATION IN EVERETT SECTION OF RESERVOIR BELOW THAT PRODUCING BACKWATER OVER CANAL WEIR

PLATE 37



PROFILE

SCALE: HOR. - NOT TO SCALE
VERT. 1" = 20'-0"

MERRIMACK VALLEY FLOOD CONTROL

HOPKINTON EVERETT RESERVOIR

WATER SURFACE PROFILES

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940 FILE NO.

PLATE 38

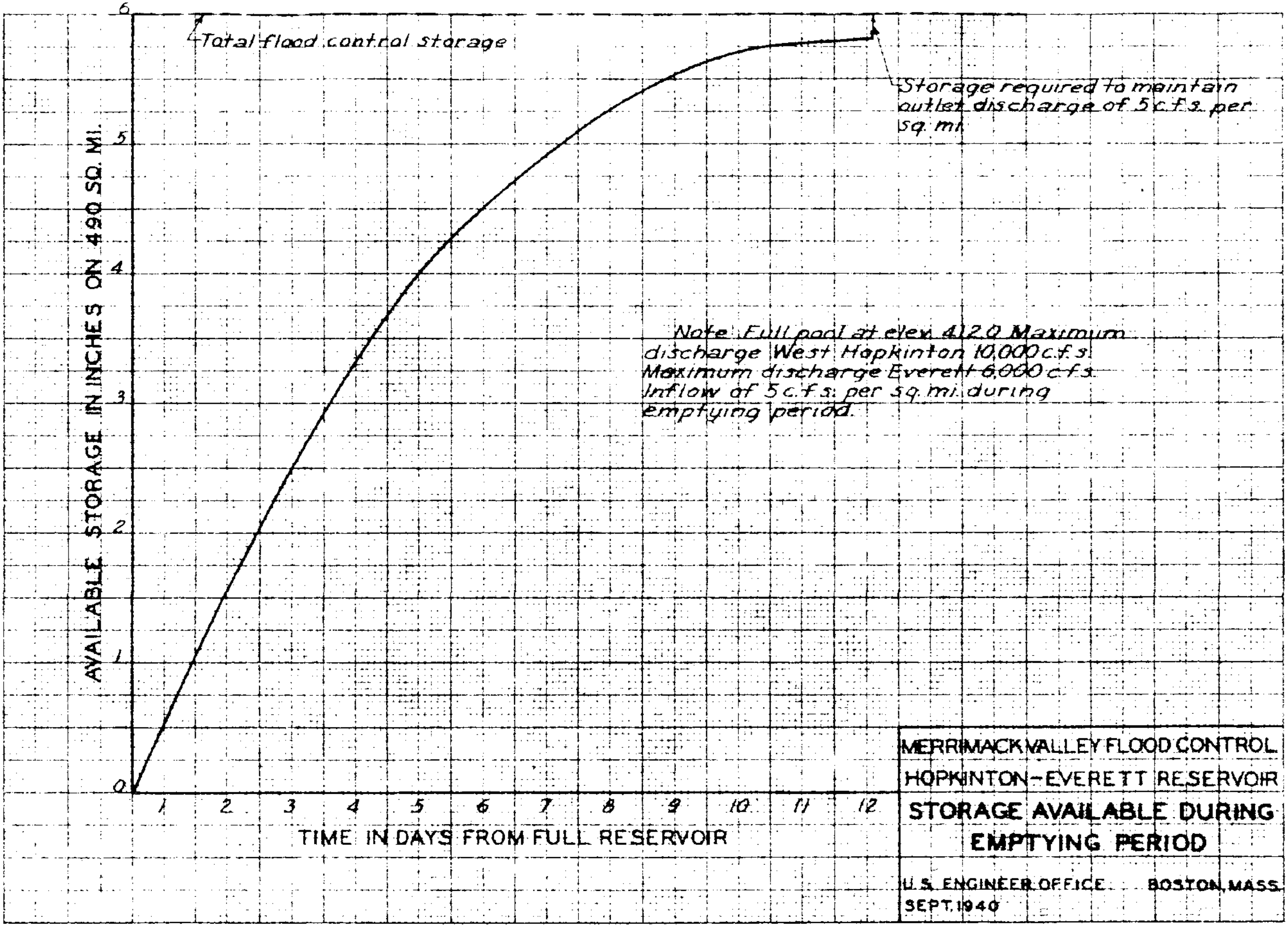
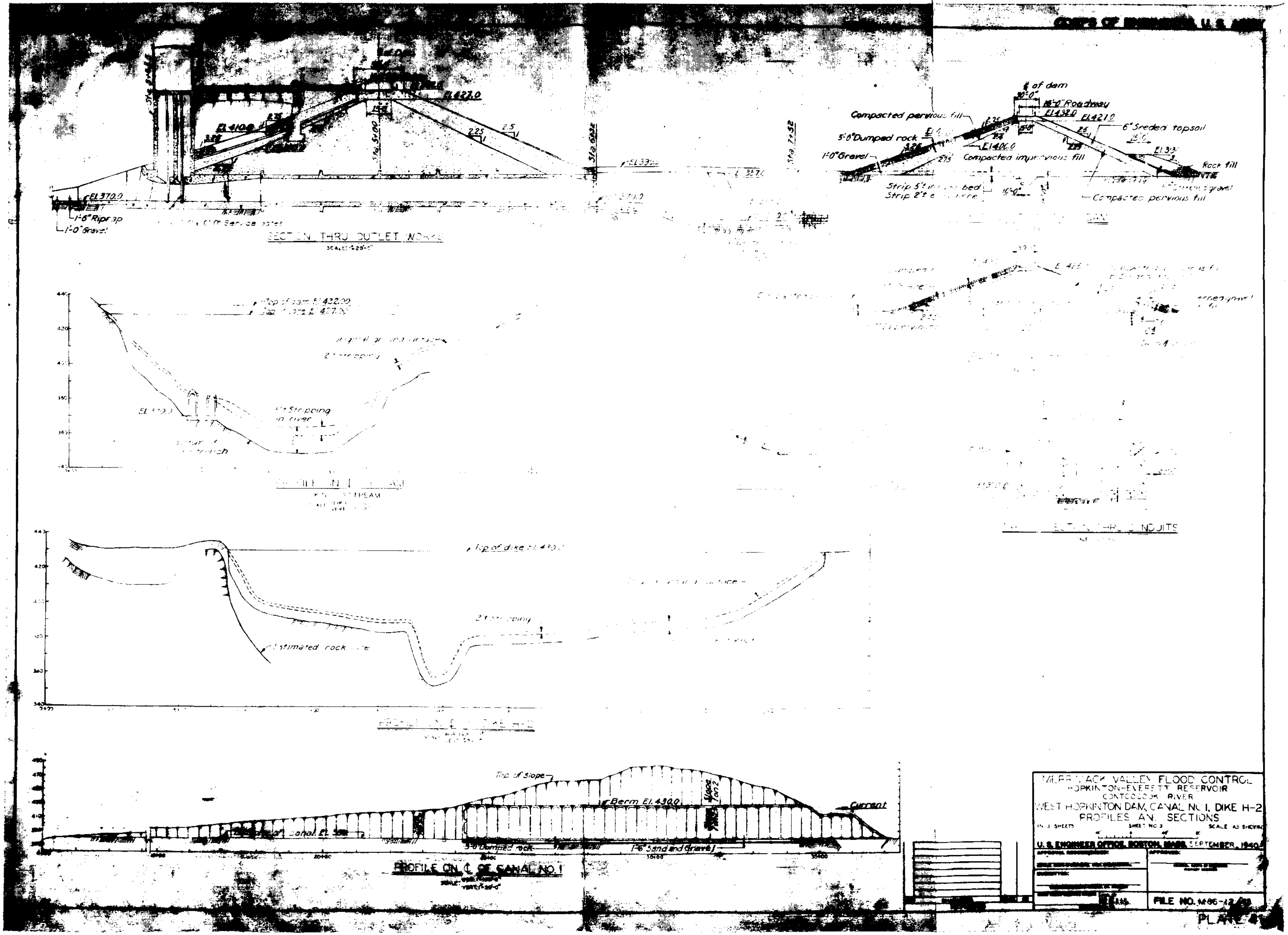
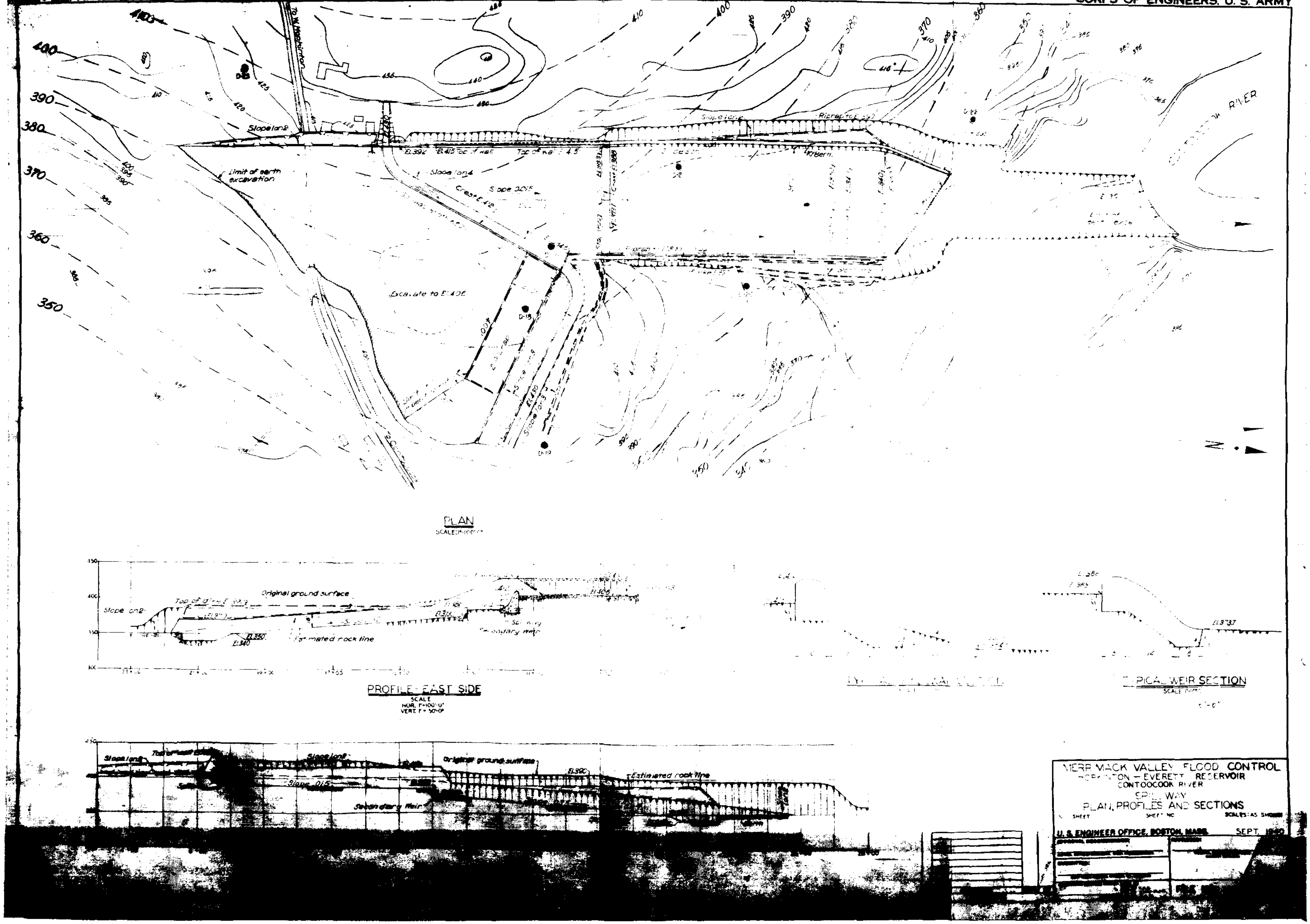

Total flood control storage
Storage required to maintain outlet discharge of 5 c.f.s. per sq. mi.
Note: Full pool at elev. 412.0 Maximum discharge West Hopkinton 10,000 c.f.s. Maximum discharge Everett 6,000 c.f.s. Inflow of 5 c.f.s. per sq. mi. during emptying period.
AVAILABLE STORAGE IN INCHES ON 490 SQ. MI.
0
1
2
3
4
5
6
1
2
3
4
5
6
7
8
9
10
11
12
TIME IN DAYS FROM FULL RESERVOIR
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
STORAGE AVAILABLE DURING EMPTYING PERIOD
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1940

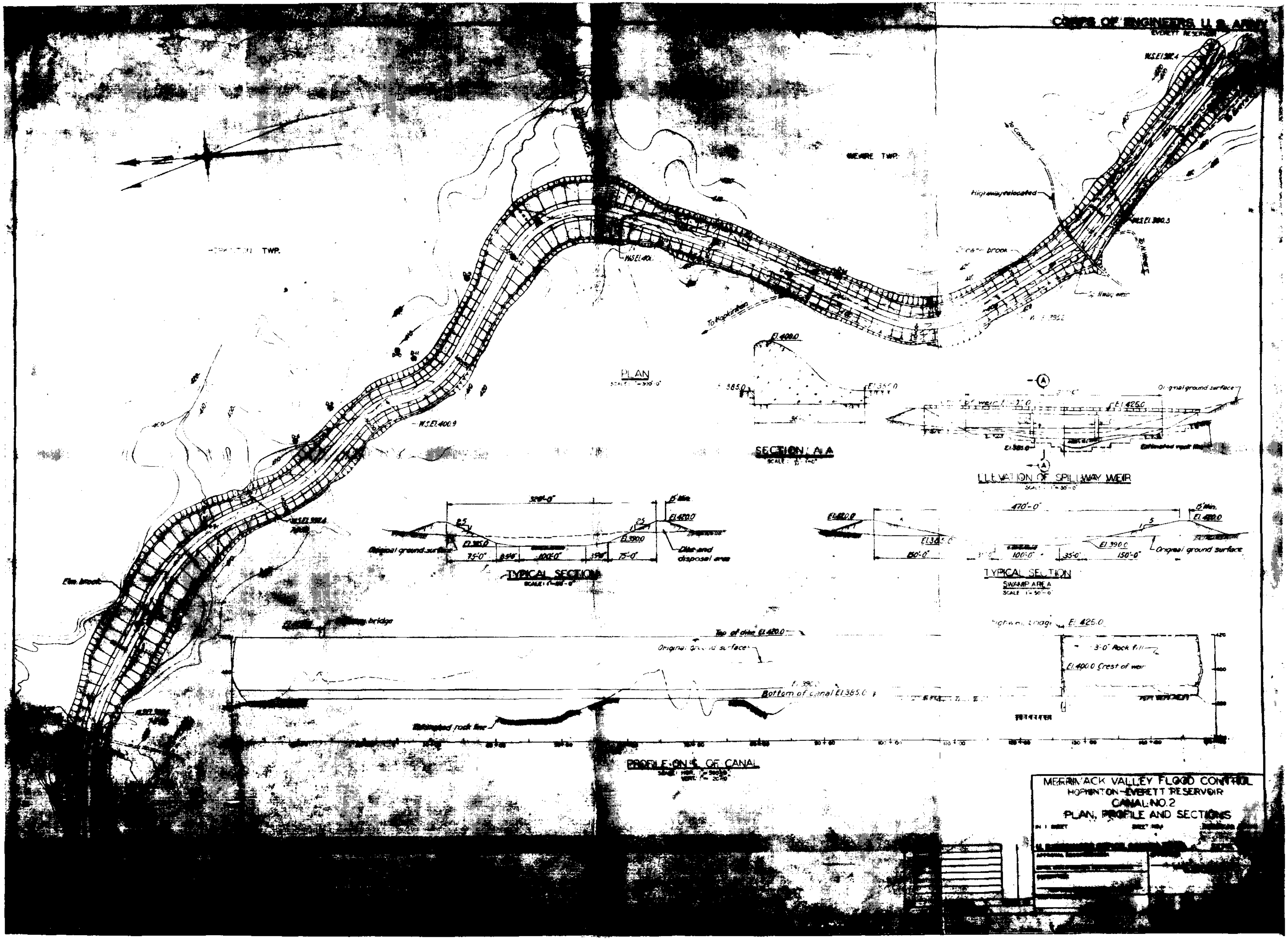
PLATE 39


CANAL NO. 1
DIKE H-2
WEST HOPKINTON DAM
20' Berm El. 490
Canal El. 360
5' Dumped rock
Top of dike 30' wide El. 430
℄ 90' Impervious blanket
℄ 90' Impervious blanket
16' Access Road
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
CONTOOCOOK RIVER
WEST HOPKINTON DAM, CANAL NO. 1, DIKE H-2
GENERAL PLAN

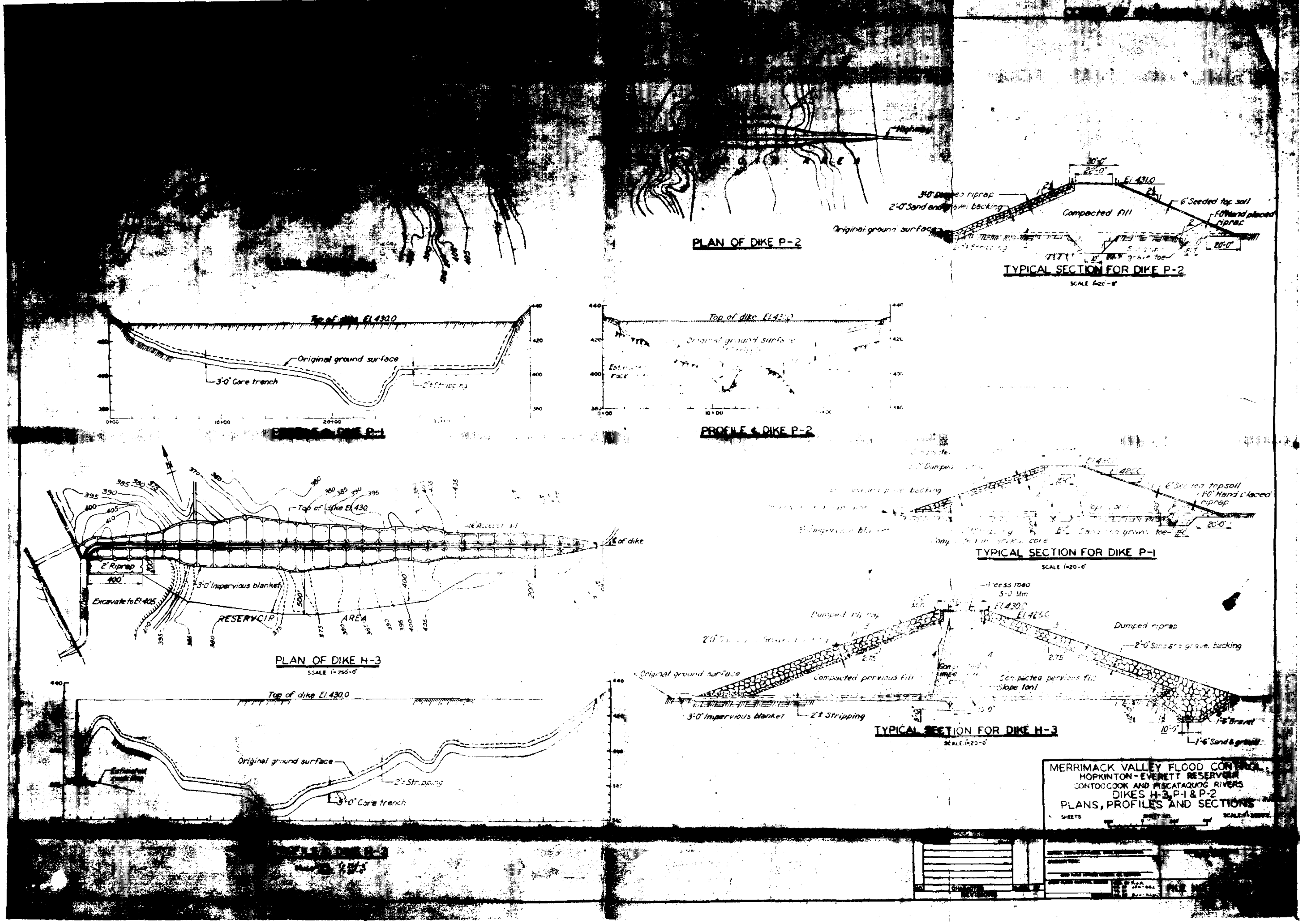

CORPS OF ENGINEERS, U.S. ARMY
El. 427.0
El. 410.0
El. 370.0
1'-6" Riprap
1'-0" Gravel
SECTION THRU OUTLET WORKS
℄ of dam
30'-0"
16'-0" Roadway
El. 438.0
El. 421.0
El. 400.0
Compacted pervious fill
5'-0" Dumped rock
1'-0" Gravel
Compacted impervious fill
6" Seeded topsoil
Rock fill
Compacted pervious fill
Strip 5' in river bed
SECTION THRU CONDUITS
Top of dam El. 422.00
Strip 2' ... in river
Top of dike El. 430.0
2' stripping
Estimated rock line
Top of slope
Berm El. 430.0
Current
5'-0" Dumped rock
1'-6" Sand and Gravel
PROFILE ON ℄ OF CANAL NO. 1
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CONTOOCOOK RIVER
WEST HOPKINTON DAM, CANAL NO. 1, DIKE H-2
PROFILES AND SECTIONS
SCALE AS SHOWN
U.S. ENGINEER OFFICE, BOSTON, MASS. SEPTEMBER, 1940
FILE NO. M66-
PLATE 4

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

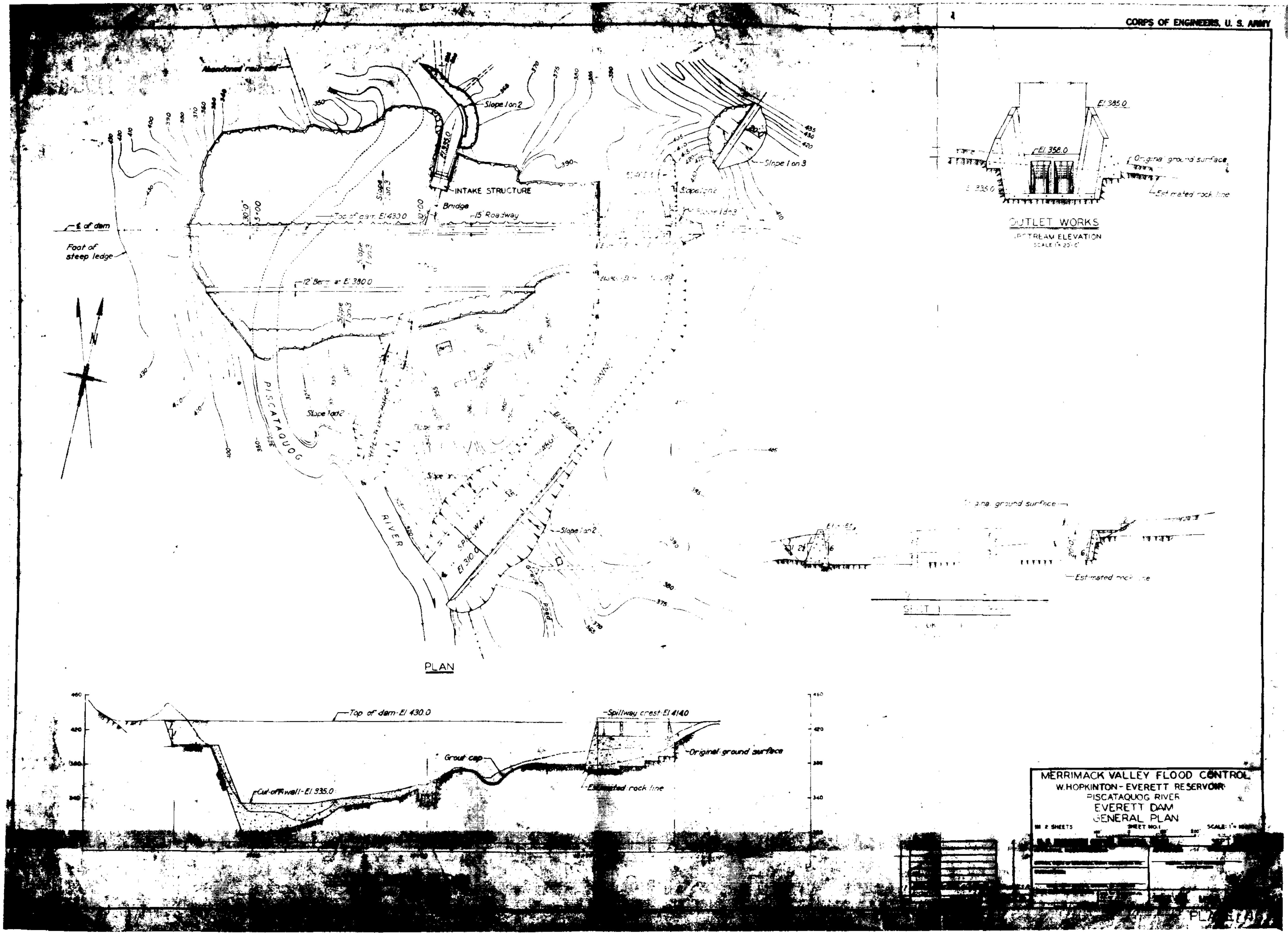


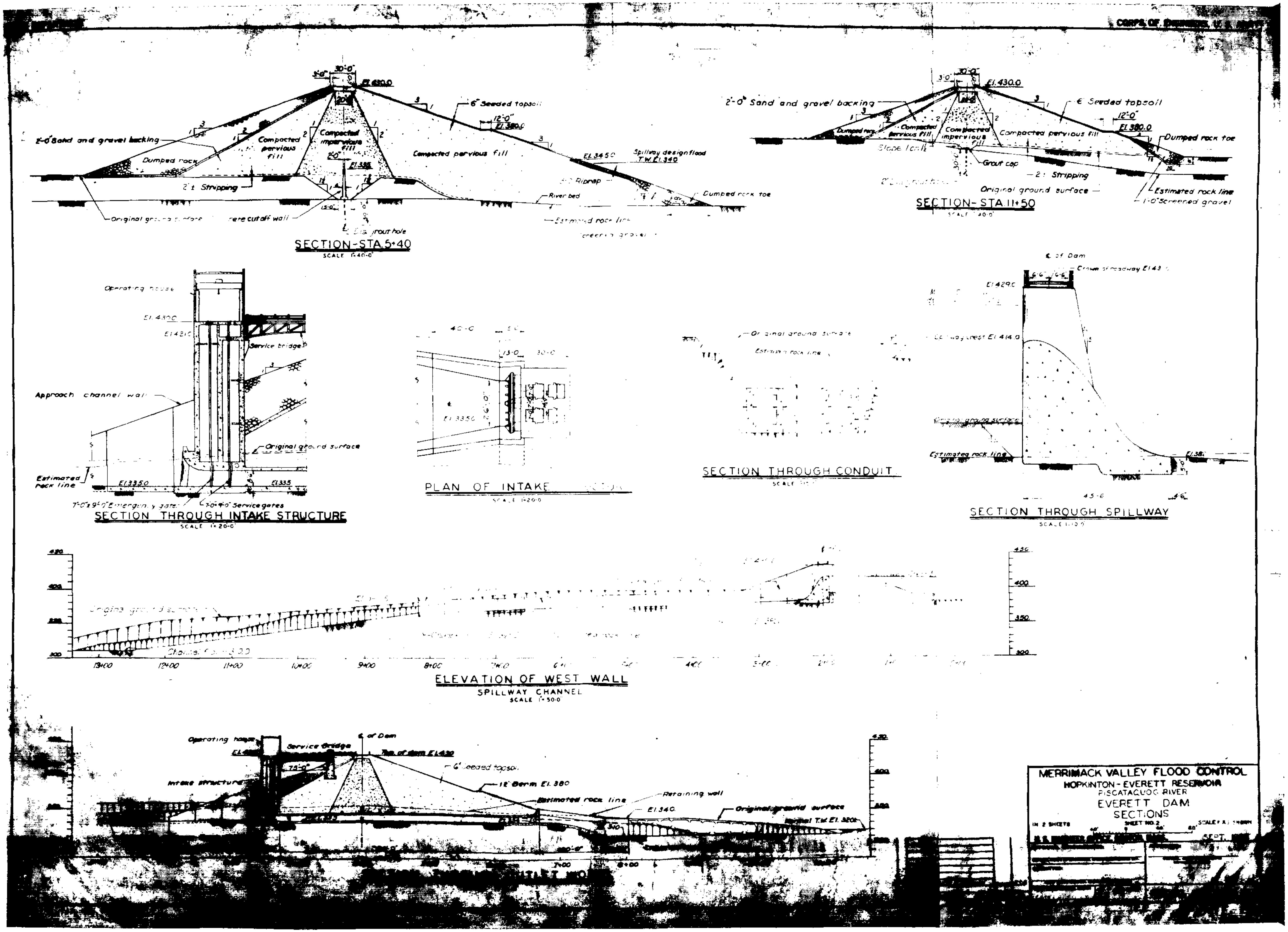


CORPS OF ENGINEERS U. S. ARMY
HOPKINTON TWP.
WEARE TWP.
Highway relocated
To Concord
To Hopkinton
Spillway weir
Elm brook
PLAN
SECTION A-A
ELEVATION OF SPILLWAY WEIR
Original ground surface
TYPICAL SECTION
TYPICAL SECTION
SWAMP AREA
Dike and disposal area
Top of dike El.420.0
Original ground surface
Bottom of canal El.385.0
Estimated rock line
Highway bridge El. 425.0
3'-0" Rock fill
El.400.0 Crest of weir
PROFILE ON ℄ OF CANAL
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CANAL NO.2
PLAN, PROFILE AND SECTIONS

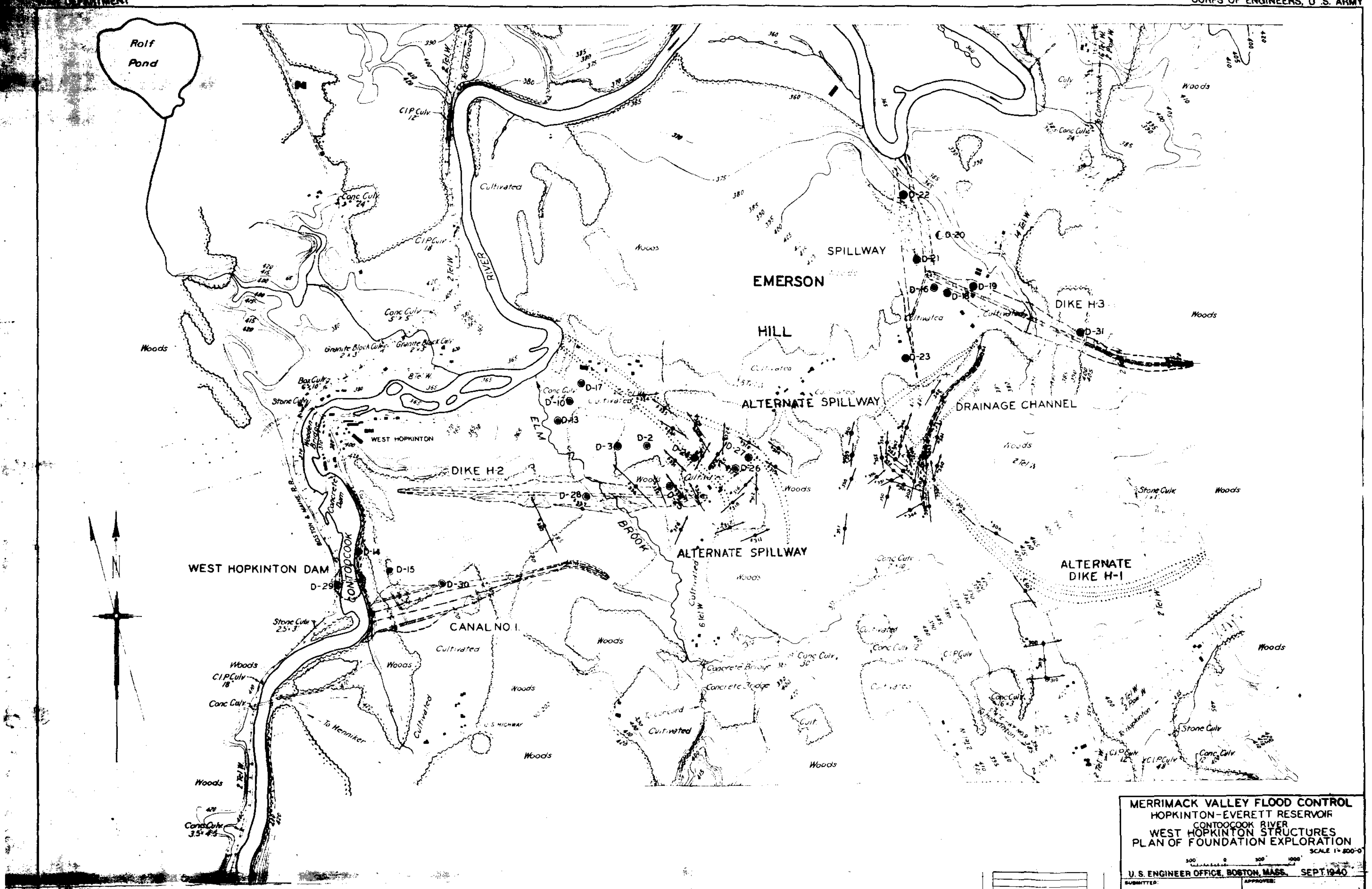


PLAN OF DIKE P-2
TYPICAL SECTION FOR DIKE P-2
Compacted fill
Original ground surface
El 431.0
6" Seeded top soil
PROFILE ℄ DIKE P-2
Top of dike El 430.0
Original ground surface
3'-0" Core trench
TYPICAL SECTION FOR DIKE P-1
6" Seeded topsoil
1'-0" Hand placed riprap
Top of dike El 430
℄ of dike
2' Riprap
400'
3'-0" Impervious blanket
Excavate to El 405
RESERVOIR
AREA
PLAN OF DIKE H-3
Dumped riprap
2'-0" Sand and gravel backing
Original ground surface
Compacted pervious fill
3'-0" Impervious blanket
2'± Stripping
TYPICAL SECTION FOR DIKE H-3
2'± Stripping
3'-0" Core trench
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CONTOOCOOK AND PISCATAQUOG RIVERS
DIKES H-3, P-1 & P-2
PLANS, PROFILES AND SECTIONS


CORPS OF ENGINEERS, U. S. ARMY
Abandoned railroad
Slope 1 on 2
Slope 1 on 3
INTAKE STRUCTURE
Bridge
15' Roadway
Top of dam El 430.0
℄ of dam
Foot of steep ledge
12' Berm at El 380.0
PISCATAQUOG
RIVER
SPILLWAY
El 310.0
Slope 1 on 2
PLAN
OUTLET WORKS
UPSTREAM ELEVATION
El 385.0
El 358.0
El 335.0
Original ground surface
Estimated rock line
Top of dam-El 430.0
Spillway crest El 414.0
Original ground surface
Grout cap
Cut-off wall-El 335.0
Estimated rock line
MERRIMACK VALLEY FLOOD CONTROL
W. HOPKINTON - EVERETT RESERVOIR
PISCATAQUOG RIVER
EVERETT DAM
GENERAL PLAN
IN 2 SHEETS
SHEET NO. 1



30'-0"
3'-0"
El. 430.0
6" Seeded topsoil
2'-0" Sand and gravel backing
Compacted pervious fill
Compacted impervious fill
Dumped rock
El. 380.0
12'-0"
El. 345.0
Spillway design flood T.W. El. 340
2' ± Stripping
Riprap
River bed
Dumped rock toe
Original ground surface
Estimated rock line
Grout hole
SECTION-STA.5+40
SCALE 1"=40'-0"
Sand and gravel backing
Compacted pervious fill
Compacted impervious fill
Dumped rock toe
Grout cap
Stripping
Original ground surface
Estimated rock line
1'-0" Screened gravel
SECTION- STA 11+50
Operating house
Service bridge
Approach channel wall
Original ground surface
Estimated rock line
El. 335.0
El. 335
Emergency gates
Service gates
SECTION THROUGH INTAKE STRUCTURE
SCALE 1"=20'-0"
PLAN OF INTAKE
SECTION THROUGH CONDUIT
Original ground surface
Estimated rock line
℄ of Dam
Crown of roadway
El. 429.0
Original ground surface
Estimated rock line
SECTION THROUGH SPILLWAY
ELEVATION OF WEST WALL
SPILLWAY CHANNEL
Original ground surface
13+00
12+00
11+00
10+00
9+00
8+00
Operating house
Service bridge
℄ of Dam
Top of dam El. 430
6" Seeded topsoil
12' Berm El. 380
Intake structure
Estimated rock line
Retaining wall
El. 340
Original ground surface
7+00
8+00
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
PISCATAQUOG RIVER
EVERETT DAM
SECTIONS
IN 2 SHEETS
SHEET NO. 2



Seismic exploration
Drill holes

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CONTOOCOOK RIVER
WEST HOPKINTON STRUCTURES
PLAN OF FOUNDATION EXPLORATION

SCALE 1" = 800'-0"

U.S. ENGINEER OFFICE, BOSTON, MASS. SEPT 1940

SUBMITTED:

APPROVAL RECOMMENDED:

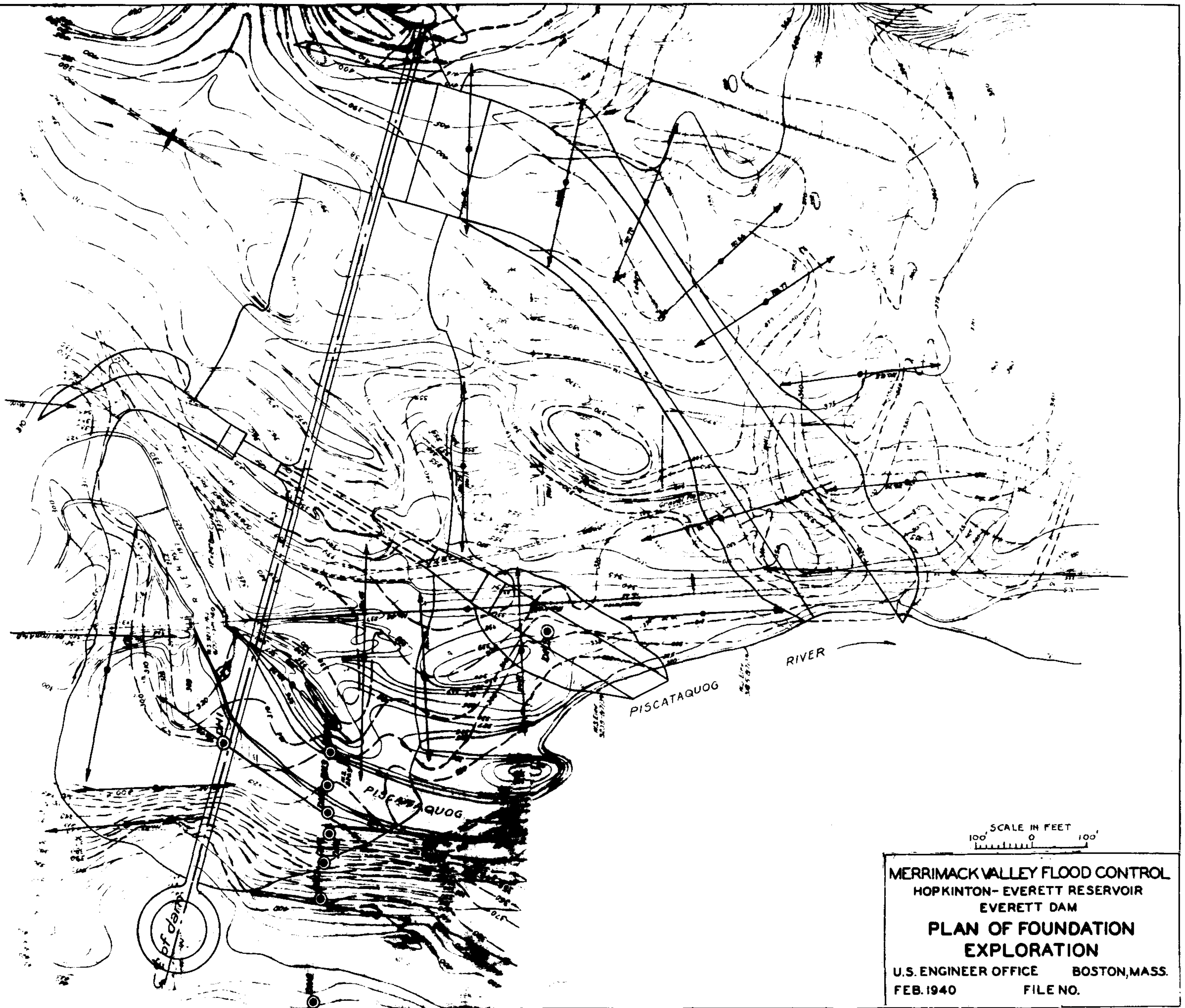
APPROVED:

NO.	CHARACTER	DATE	BY

REVISIONS

FILE NO.

PLATE



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON- EVERETT RESERVOIR
EVERETT DAM

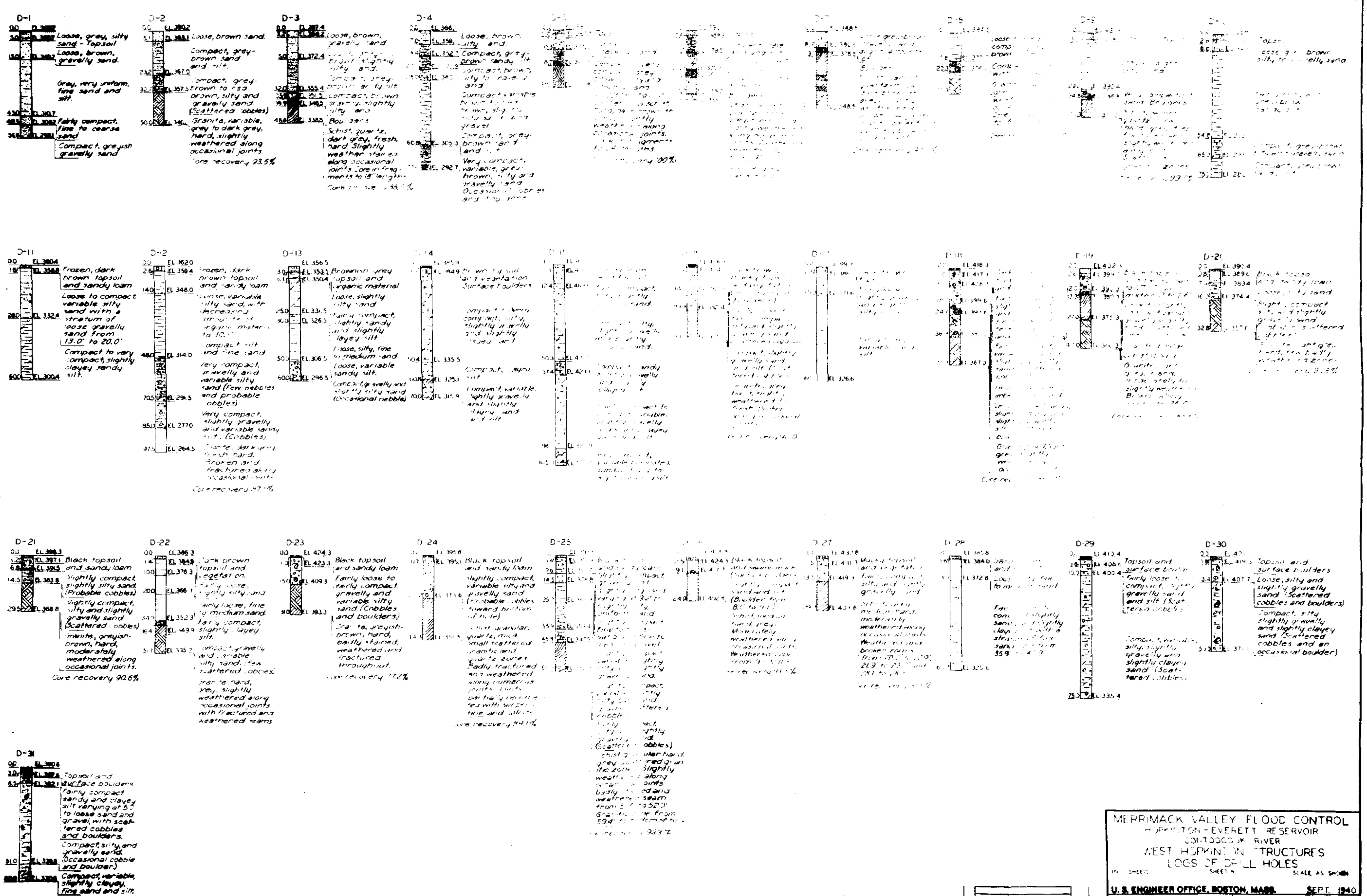
PLAN OF FOUNDATION EXPLORATION

U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 48

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR
CONTOOCOOK RIVER
WEST HOPKINTON STRUCTURES
LOGS OF DRILL HOLES

IN SHEETS | SHEET N | SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. SEPT. 1940

APPROVAL RECOMMENDED: | APPROVED:

SUBMITTED:

REVISIONS | DATE | BY

FILE NO. M 66-12/49

PLATE 49

D-1

0.0	EL.396.2	
2.0	EL.394.2	*Topsoil*
7.9	EL.388.3	*Fairly loose, brown gravelly sand, varying amounts of silt, occasional stone.*
27.9	EL.368.3	*Granite-gneiss, fresh, hard, coarse-grained.*

Core recovery 93.0%

D-2

0.0	EL.397.6	
3.8	EL.393.8	*Topsoil-organic material.*
9.9	EL.387.7	*Compact, gravelly, very silty sand. Cobbles.*
15.8	EL.381.8	*Very compact, slightly gravelly and clayey silty sand.*
23.8	EL.373.8	*Gneiss, porphyritic, coarse, grey, medium hard, slightly weathered to fresh. Badly fractured.*

Core recovery 68.8%

D-3

0.0	EL.399.4	
2.5	EL.396.9	*Topsoil*
5.8	EL.393.6	*Loose to compact, silty gravelly sand.*
17.2	EL.382.2	*Compact, silty sand and gravel.*
20.0	EL.379.4	*Compact, gravelly sand and silt.*

D-4

0.0	EL.404.0	
4.0	EL.400.0	*Organic material-muck.*
10.2	EL.393.8	*Fairly compact, sandy to clayey silt.*
25.5	EL.378.5	*Compact, gravelly, slightly silty sand. Occasional cobbles.*

D-5

0.0	EL.411.8	
2.0	EL.409.8	*Topsoil.*
9.0	EL.402.8	*Compact, sand and gravel.*
13.9	EL.397.9	*Very compact, silty sand and gravel.*
17.3	EL.394.5	*Very compact, silty sand and soft, very badly weathered rock or boulders.*
22.9	EL.388.9	
25.5	EL.386.3	*Weathered quartz mica schist.*
		Soft, badly weathered garnet-mica schist.

Core recovery 35.4%

D-6

0.0	EL.399.3	
5.0	EL.394.3	*Organic material and muck.*
15.6	EL.383.7	*Fairly compact, to compact, gravelly sand.*
20.3	EL.379.3	*Loose, slightly gravelly and silty sand.*

D-7

0.0	EL.397.7	
7.0	EL.390.7	*Organic material and muck.*
20.0	EL.377.7	*Loose, gravelly, slightly silty sand.*

D-8

0.0	EL.395.9	
20.0	EL.375.9	*Very loose, organic material.*
25.0	EL.370.9	*Fairly compact, gravelly, slightly silty sand.*

D-9

0.0	EL.414.0	
3.0	EL.411.0	*Topsoil*
10.0	EL.404.0	*Compact, fine to coarse sand.*
18.6	EL.395.4	*Compact, gravelly to silty sand.*
28.4	EL.385.6	*Compact, variable, gravelly, slightly silty sand.*
29.0	EL.385.0	*Badly weathered mica schist.*

D-10

0.0	EL.407.7	
2.0	EL.405.7	*Dark brown topsoil and organic material.*
8.0	EL.399.7	*Fairly compact, slightly silty and slightly gravelly sand. (Cobbles)*
14.5	EL.393.2	*Completely disintegrated mica-schist probably in situ.*
17.8	EL.389.9	*Granite, grey, hard, fresh. Core in fragments to 6" lengths.*
23.0	EL.384.7	*Schist, dark grey, medium hard, slightly weathered along occasional joints. Core in fragments to 2" lengths.*

Core recovery 70.0%

D-11

0.0	EL.406.0	
1.0	EL.405.0	*Topsoil and organic material*
9.0	EL.397.0	*Fairly loose, variable, gravelly, slightly silty sand.*
17.0	EL.389.0	*Schist, grey, hard to medium hard, slightly weathered to fresh. Core moderately broken.*

Core recovery 81.0%

D-12

0.0	EL.408.9	
2.0	EL.406.9	*Light brown topsoil and vegetation.*
12.5	EL.396.4	*Loose to fairly compact, gravelly and slightly silty sand. (Cobbles)*
20.5	EL.388.4	*Schist, greyish-brown, medium hard, weathered along numerous joints. Core in fragments to 4" lengths.*

Core recovery 73.8%

D-13

0.0	EL.418.3	
2.0	EL.416.3	*Brown topsoil and vegetation.*
15.0	EL.403.3	*Loose to fairly compact, variable silty and gravelly sand. (Cobbles)*
25.4	EL.392.9	*Compact, slightly gravelly sand and silt. (Occasional cobbles.)*
33.6	EL.384.7	*Compact, silty sand with varying amounts of gravel.*

D-14

0.0	EL.415.5	
2.0	EL.413.5	*Compact, light brown topsoil and vegetation.*
5.6	EL.409.9	*Compact, silty, slightly gravelly sand.*
25.5	EL.390.0	*Gneiss, coarse grained, grey very slightly weathered along fracture planes and joints. Two small inclusions of schist. Core in fragments to 3" lengths.*

Core recovery 79.0%

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

CANAL NO. 2
LOG OF DRILL HOLES

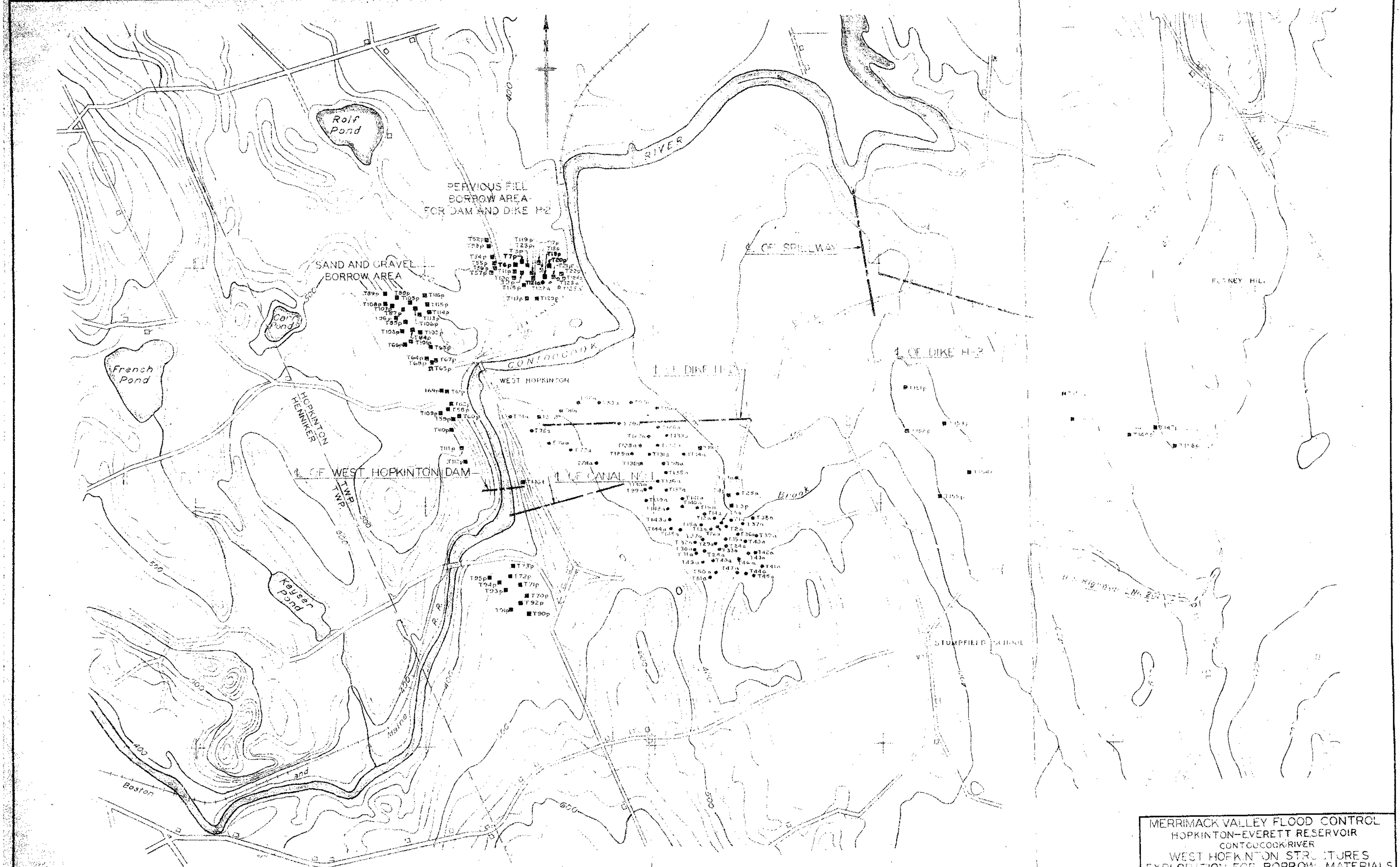
U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPTEMBER, 1940 FILE NO.

MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
PISCATAQUOG RIVER
EVERETT DAM
LOG OF DRILL HOLES

PLATE 51

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CONTOOCOOK RIVER
WEST HOPKINTON STRUCTURES
EXPLORATION FOR BORROW MATERIALS
IN 1 SHEET SHEET NO. 1 SCALE: 1"=600 FT.
600 0 600 1200
U. S. ENGINEER OFFICE, BOSTON, MASS. SEPT. 1940

APPROVAL RECOMMENDED:
CAPTAIN, CORPS OF ENGINEERS, CHIEF, ENGINEERING DIV.
SUBMITTED:
CHIEF FLOOD CONTROL DIVISION, ENGINEER
CHIEF FLOOD CONTROL DESIGN ENGINEER
DES. BY
DR. BY
TR. BY J.T.H.
CK. BY
APPROVED:
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

NO.	CHARACTER	DATE	BY

REVISIONS

FILE NO. M66-12/45

PLATE 52

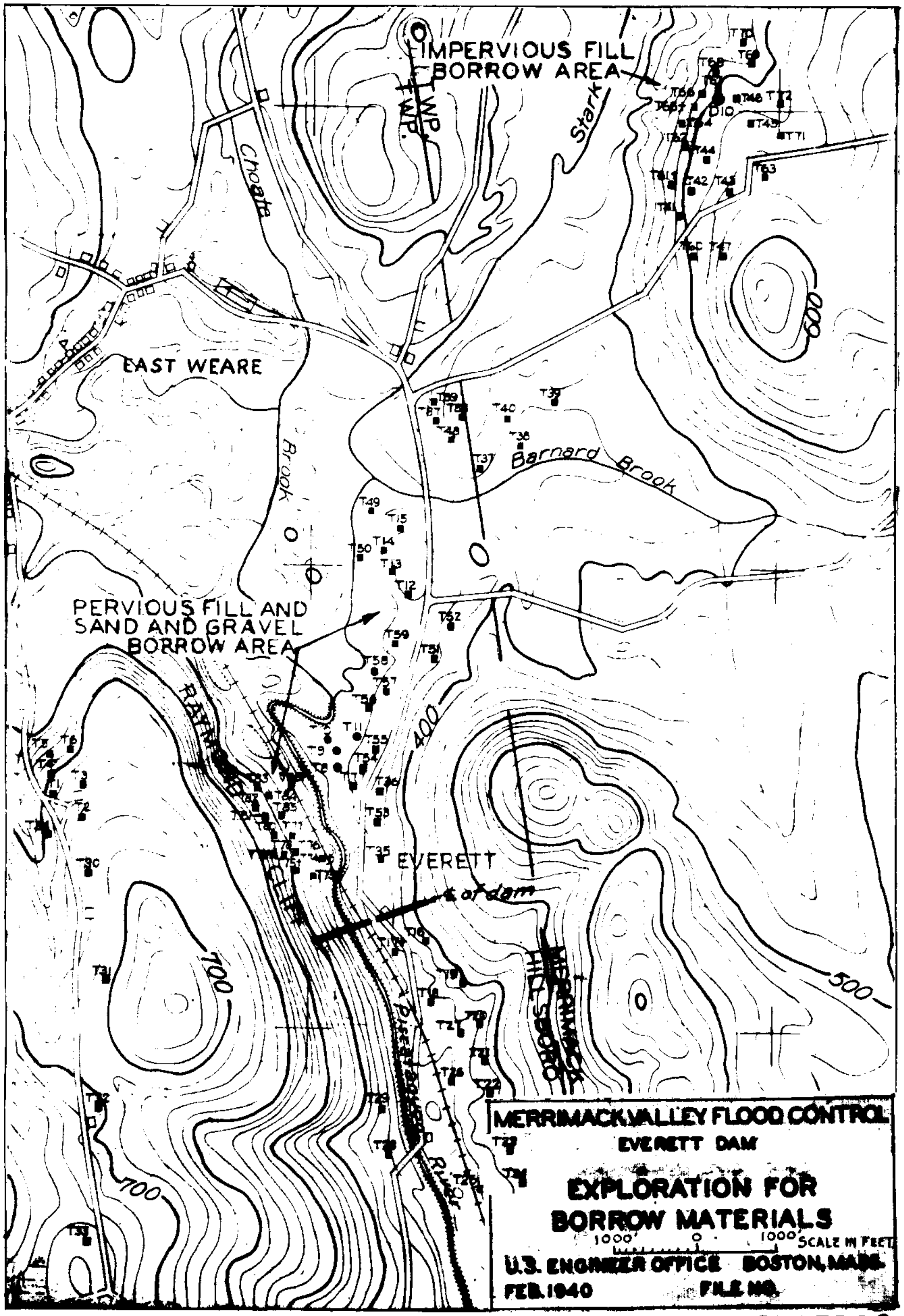

IMPERVIOUS FILL
BORROW AREA
TWP.
TWP.
Stark
Choate
EAST WEARE
Brook
Barnard Brook
600
PERVIOUS FILL AND
SAND AND GRAVEL
BORROW AREA
400
EVERETT
℄ of dam
700
500
700
HILLSBORO
MERRIMACK
Piscataquog River
MERRIMACK VALLEY FLOOD CONTROL
EVERETT DAM
EXPLORATION FOR
BORROW MATERIALS
1000' 0 1000' SCALE IN FEET
U.S. ENGINEER OFFICE BOSTON, MASS.
FEB. 1940 FILE NO.

PLATE 53

WAR DEPARTMENT

CORPS OF ENGINEERS

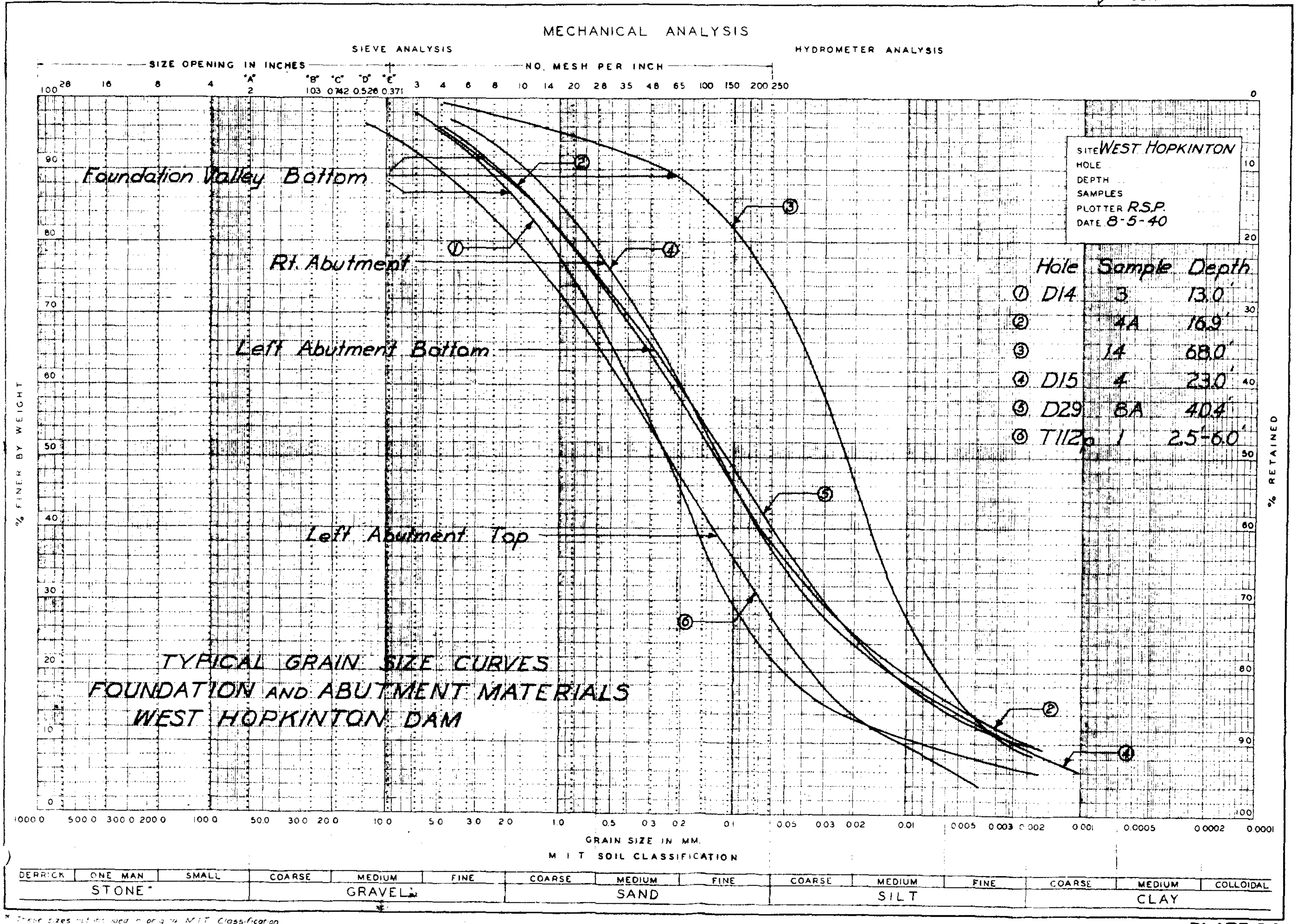
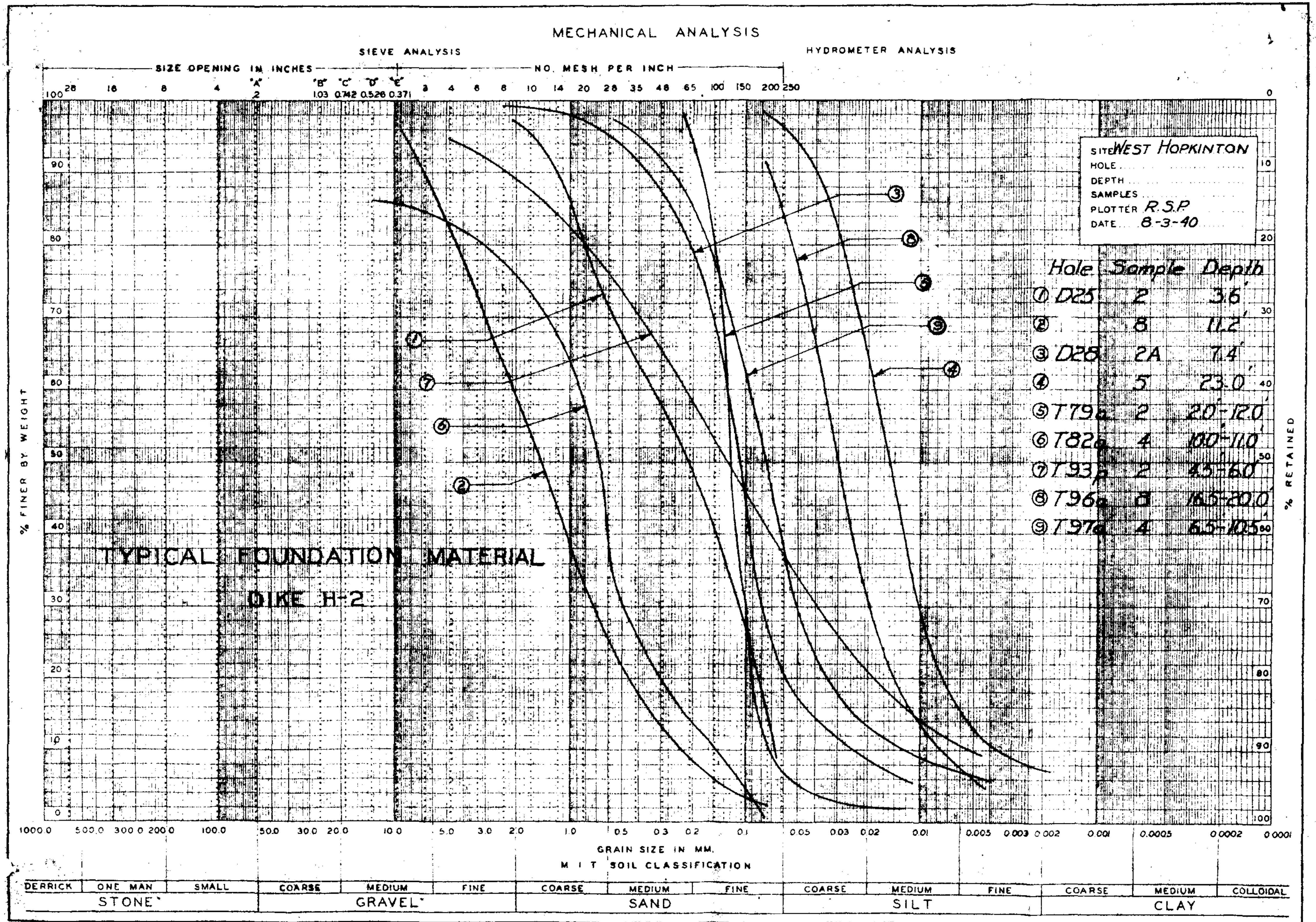
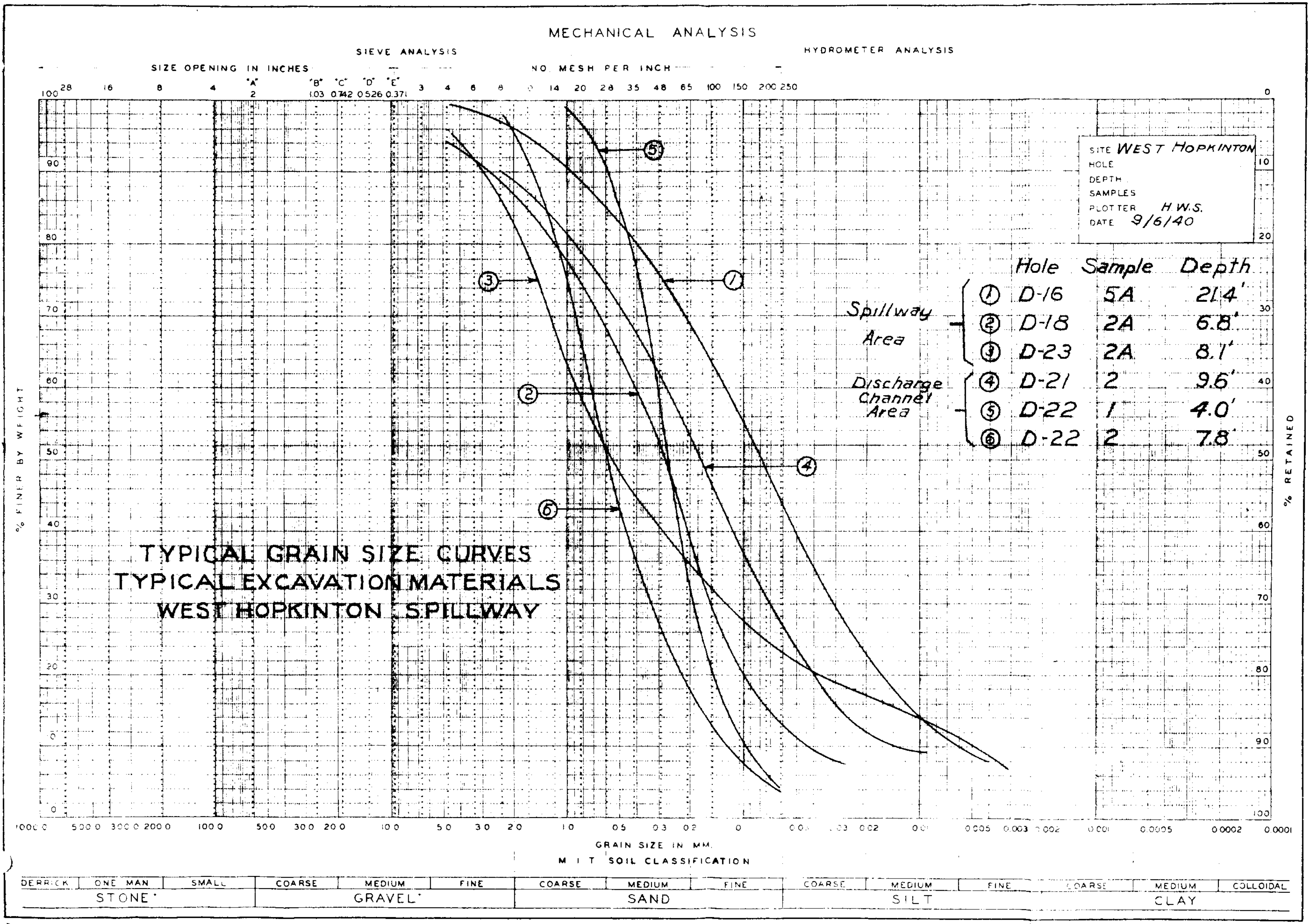
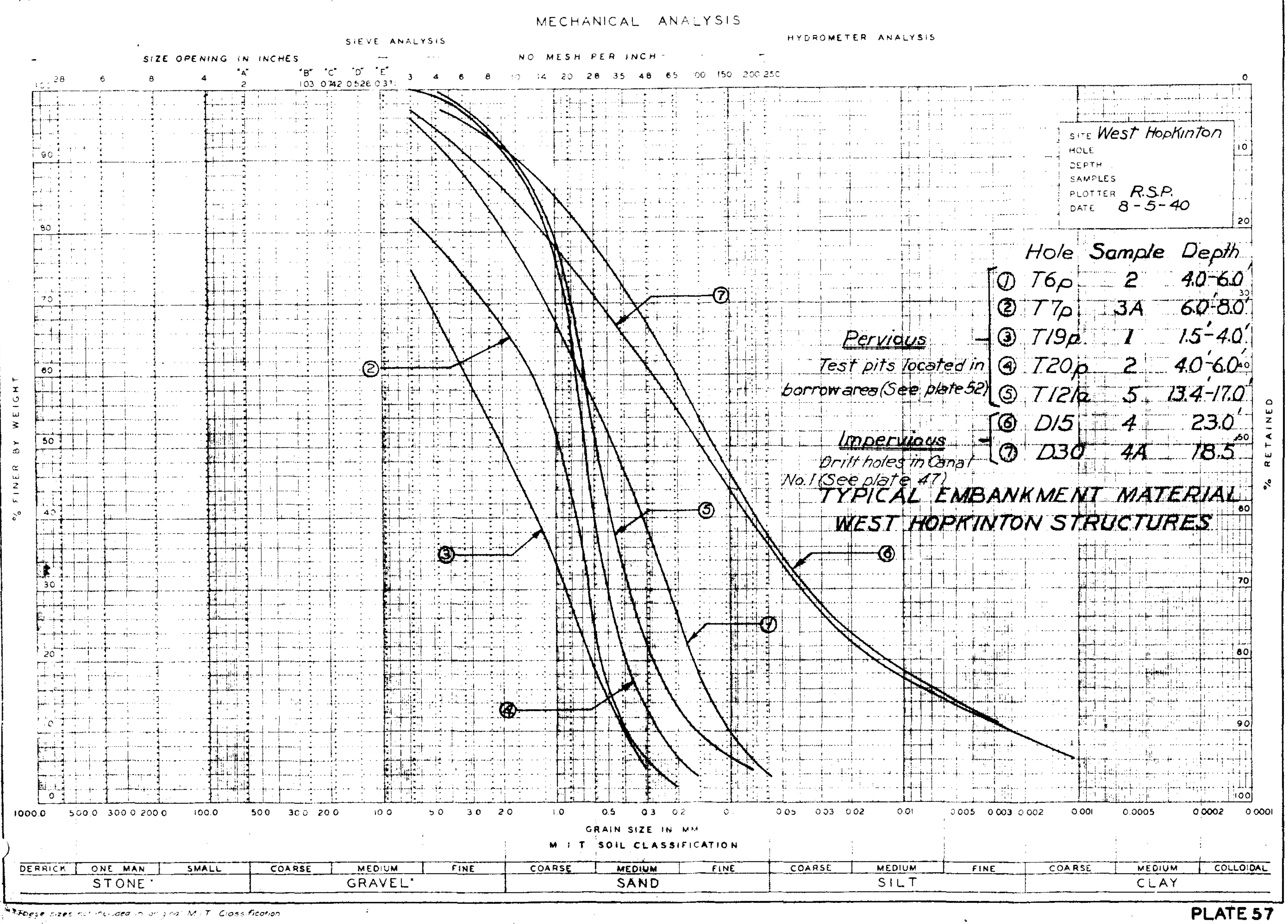


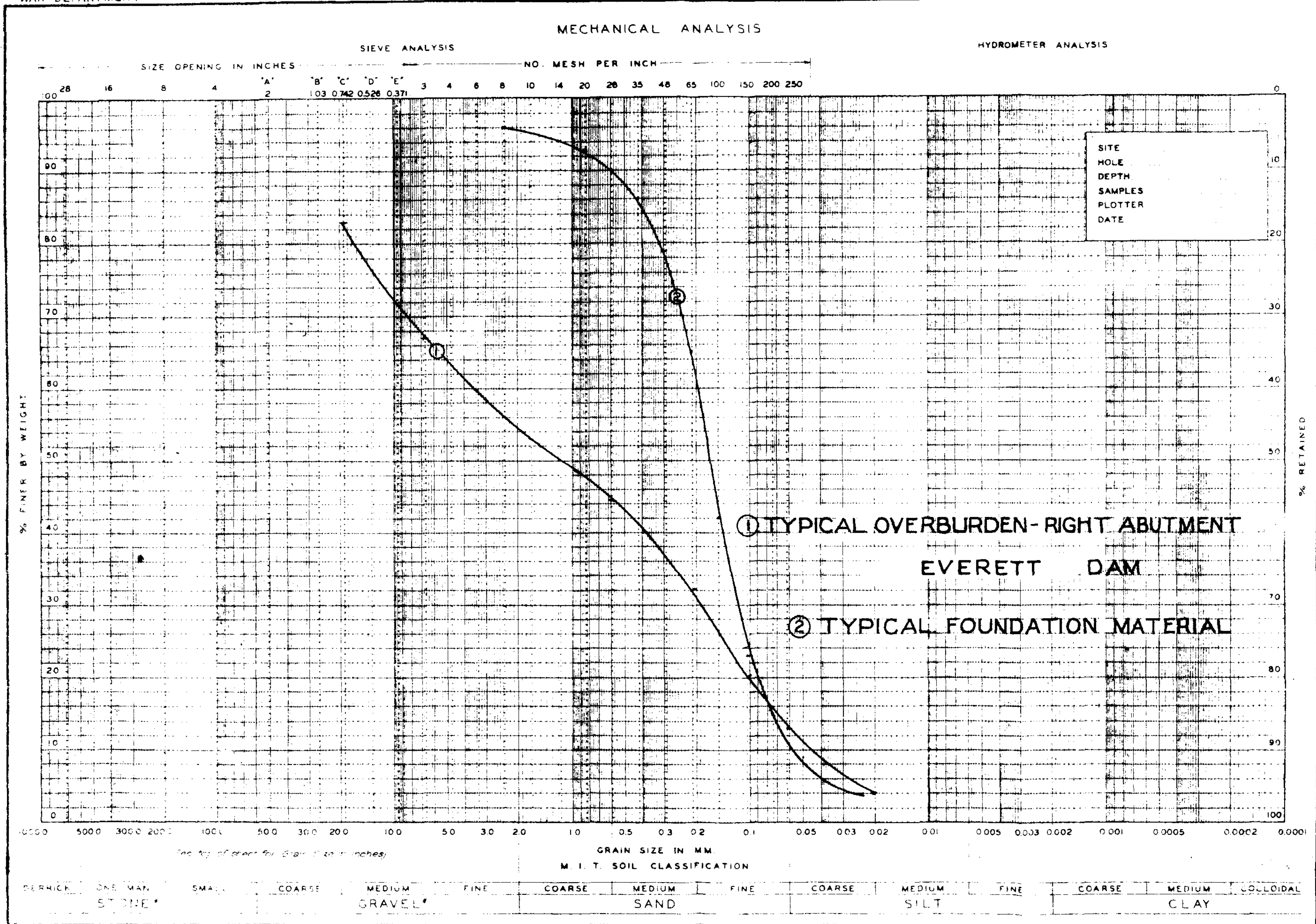
PLATE 54

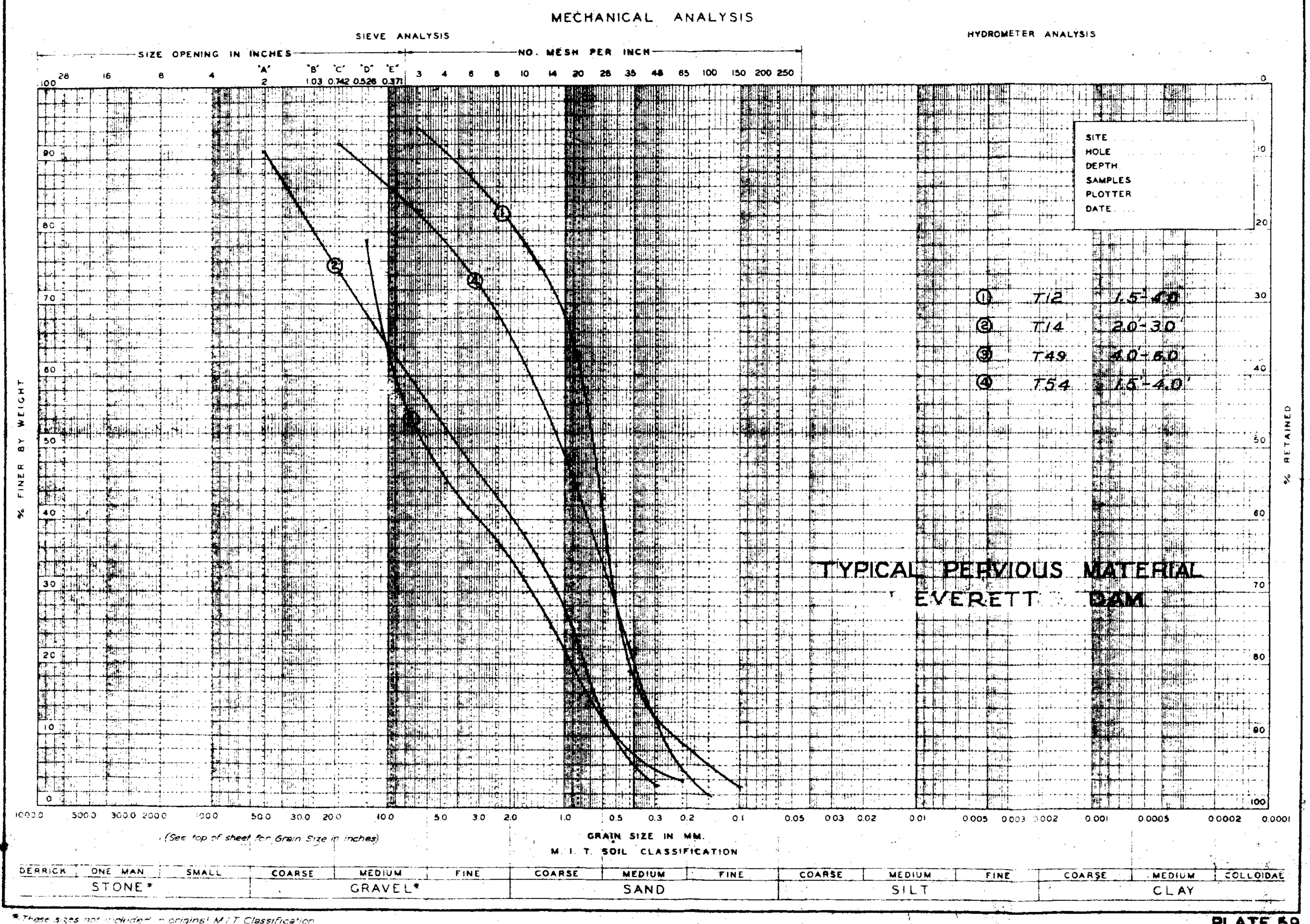


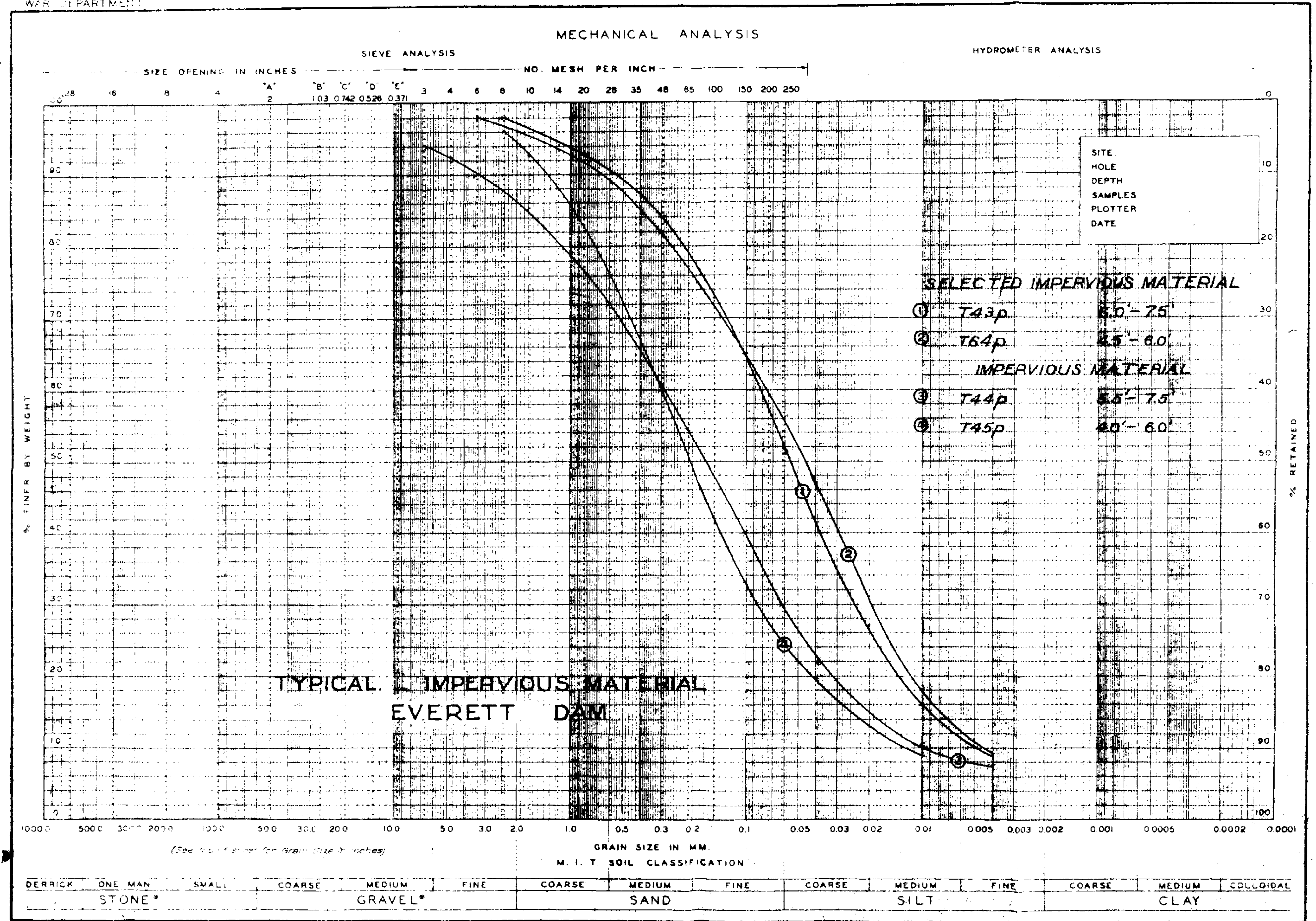
*These sizes not included in original M.I.T. Classification


MECHANICAL ANALYSIS
SIEVE ANALYSIS
HYDROMETER ANALYSIS
SIZE OPENING IN INCHES
NO. MESH PER INCH
SITE WEST HOPKINTON
HOLE
DEPTH
SAMPLES
PLOTTER H.W.S.
DATE 9/6/40
Hole Sample Depth
Spillway Area
① D-16 5A 21.4'
② D-18 2A 6.8'
③ D-23 2A 8.1'
Discharge Channel Area
④ D-21 2 9.6'
⑤ D-22 1 4.0'
⑥ D-22 2 7.8'
TYPICAL GRAIN SIZE CURVES
TYPICAL EXCAVATION MATERIALS
WEST HOPKINTON SPILLWAY
% FINER BY WEIGHT
% RETAINED
GRAIN SIZE IN MM.
M.I.T. SOIL CLASSIFICATION
DERRICK
ONE MAN
SMALL
COARSE
MEDIUM
FINE
STONE*
GRAVEL*
SAND
SILT
CLAY
COLLOIDAL

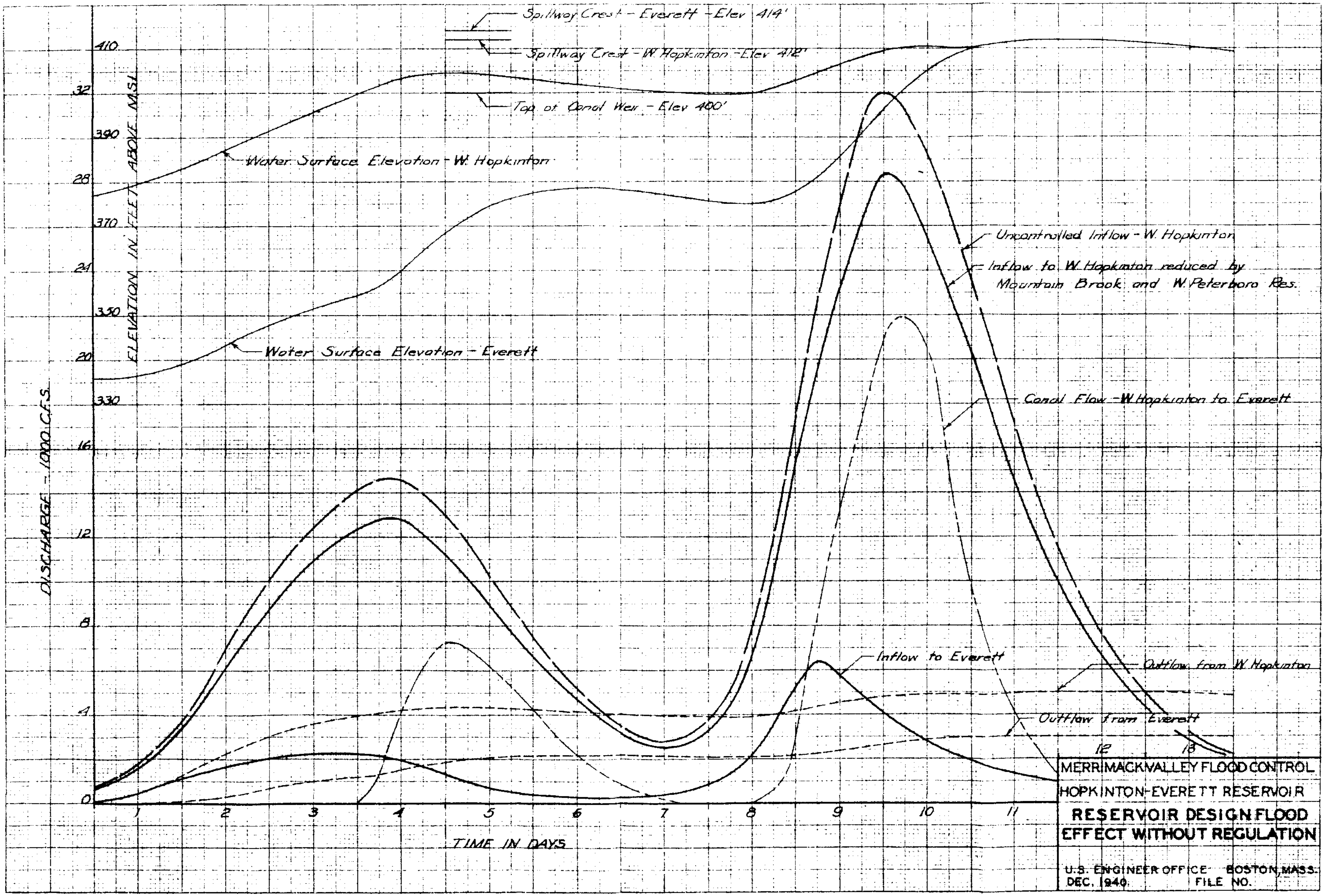

MECHANICAL ANALYSIS
SIEVE ANALYSIS
HYDROMETER ANALYSIS
SIZE OPENING IN INCHES
NO MESH PER INCH
SITE West Hopkinton
HOLE
DEPTH
SAMPLES
PLOTTER R.S.P.
DATE 8-5-40
Hole Sample Depth
① T6p 2 4.0'-6.0'
② T7p 3A 6.0'-8.0'
③ T19p 1 1.5'-4.0'
④ T20p 2 4.0'-6.0'
⑤ T12la 5 13.4'-17.0'
⑥ D15 4 23.0'
⑦ D30 4A 18.5'
Pervious
Test pits located in borrow area (See plate 52)
Impervious
Drift holes in Canal No. 1 (See plate 47)
TYPICAL EMBANKMENT MATERIAL
WEST HOPKINTON STRUCTURES
% FINER BY WEIGHT
% RETAINED
GRAIN SIZE IN MM
M I T SOIL CLASSIFICATION
DERRICK
ONE MAN
SMALL
COARSE
MEDIUM
FINE
STONE*
GRAVEL*
SAND
SILT
CLAY
COLLOIDAL
*These sizes not included in original M.I.T. Classification


MECHANICAL ANALYSIS
SIEVE ANALYSIS
HYDROMETER ANALYSIS
SIZE OPENING IN INCHES
NO. MESH PER INCH
SITE
HOLE
DEPTH
SAMPLES
PLOTTER
DATE
% FINER BY WEIGHT
% RETAINED
① TYPICAL OVERBURDEN - RIGHT ABUTMENT
EVERETT DAM
② TYPICAL FOUNDATION MATERIAL
GRAIN SIZE IN MM.
M. I. T. SOIL CLASSIFICATION
COARSE
MEDIUM
FINE
COARSE
MEDIUM
FINE
COARSE
MEDIUM
FINE
COARSE
MEDIUM
COLLOIDAL
STONE*
GRAVEL*
SAND
SILT
CLAY


MECHANICAL ANALYSIS
SIEVE ANALYSIS
HYDROMETER ANALYSIS
SIZE OPENING IN INCHES
NO. MESH PER INCH
% FINER BY WEIGHT
% RETAINED
SITE
HOLE
DEPTH
SAMPLES
PLOTTER
DATE
① T12 1.5'-4.0'
② T14 2.0'-3.0'
③ T49 4.0'-6.0'
④ T54 1.5'-4.0'
TYPICAL PERVIOUS MATERIAL
EVERETT DAM
(See top of sheet for Grain Size in inches)
GRAIN SIZE IN MM.
M. I. T. SOIL CLASSIFICATION
DERRICK ONE MAN SMALL
STONE*
COARSE MEDIUM FINE
GRAVEL*
COARSE MEDIUM FINE
SAND
COARSE MEDIUM FINE
SILT
COARSE MEDIUM COLLOIDAL
CLAY
* These sizes not included in original M.I.T. Classification



* These sizes not included in original M.I.T. Classification

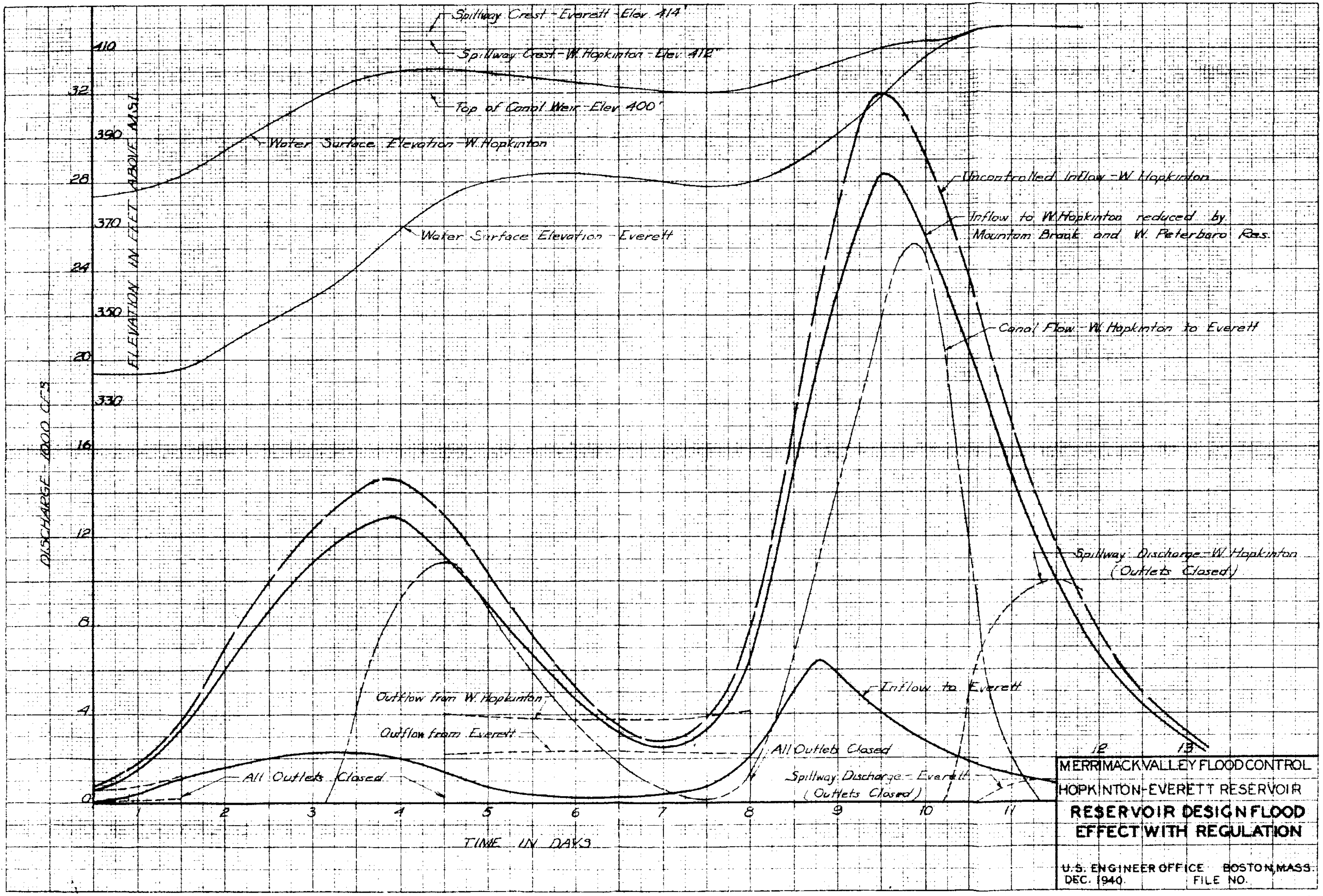


MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR

RESERVOIR DESIGN FLOOD
EFFECT WITHOUT REGULATION

U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940 FILE NO.

PLATE 61



MERRIMACK VALLEY FLOOD CONTROL

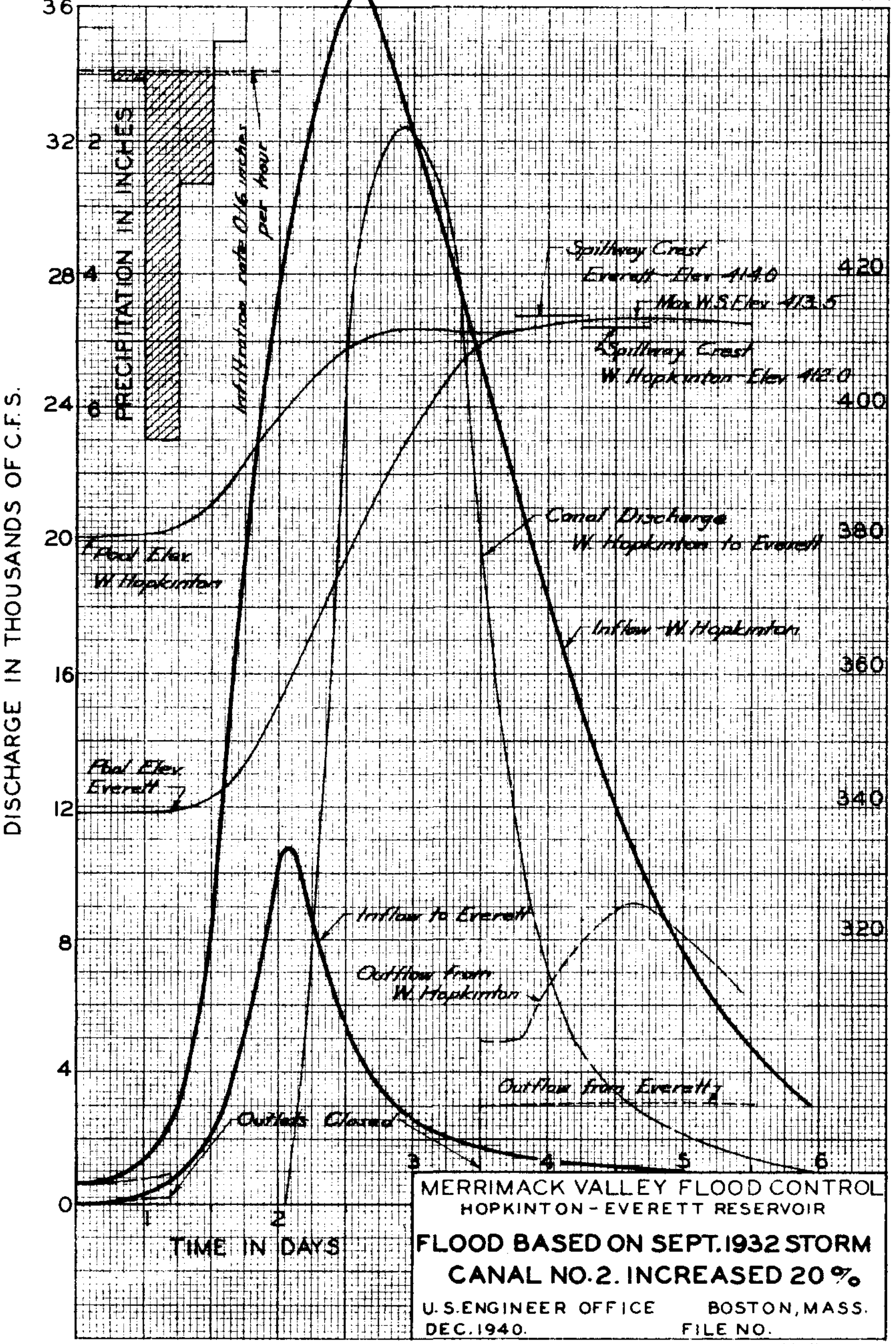
HOPKINTON-EVERETT RESERVOIR

RESERVOIR DESIGN FLOOD
EFFECT WITH REGULATION

U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940. FILE NO.

PLATE 62

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON - EVERETT RESERVOIR

FLOOD BASED ON SEPT. 1932 STORM
CANAL NO. 2. INCREASED 20 %

U. S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940. FILE NO.

PLATE 63

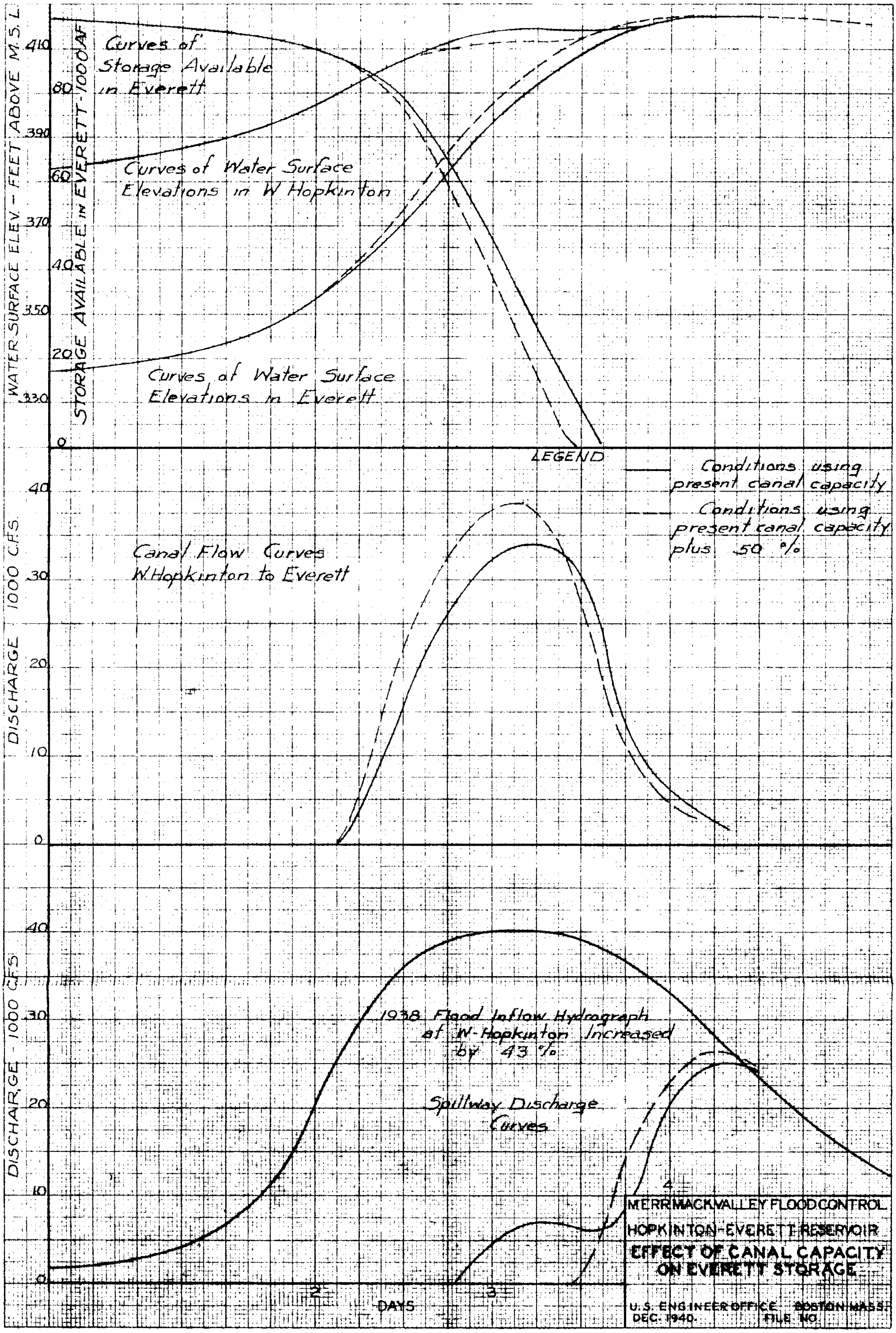

Curves of Storage Available in Everett
Curves of Water Surface Elevations in W Hopkinton
Curves of Water Surface Elevations in Everett
WATER SURFACE ELEV. - FEET ABOVE M.S.L.
STORAGE AVAILABLE IN EVERETT - 1000 AF
410
390
370
350
330
80
60
40
20
0
LEGEND
Conditions using present canal capacity
Conditions using present canal capacity plus 50 %
Canal Flow Curves W. Hopkinton to Everett
DISCHARGE 1000 C.F.S.
40
30
20
10
0
1938 Flood Inflow Hydrograph at W-Hopkinton Increased by 43 %
Spillway Discharge Curves
DISCHARGE - 1000 C.F.S.
2
3
4
DAYS
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
EFFECT OF CANAL CAPACITY ON EVERETT STORAGE
U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940.
FILE NO.

PLATE 64

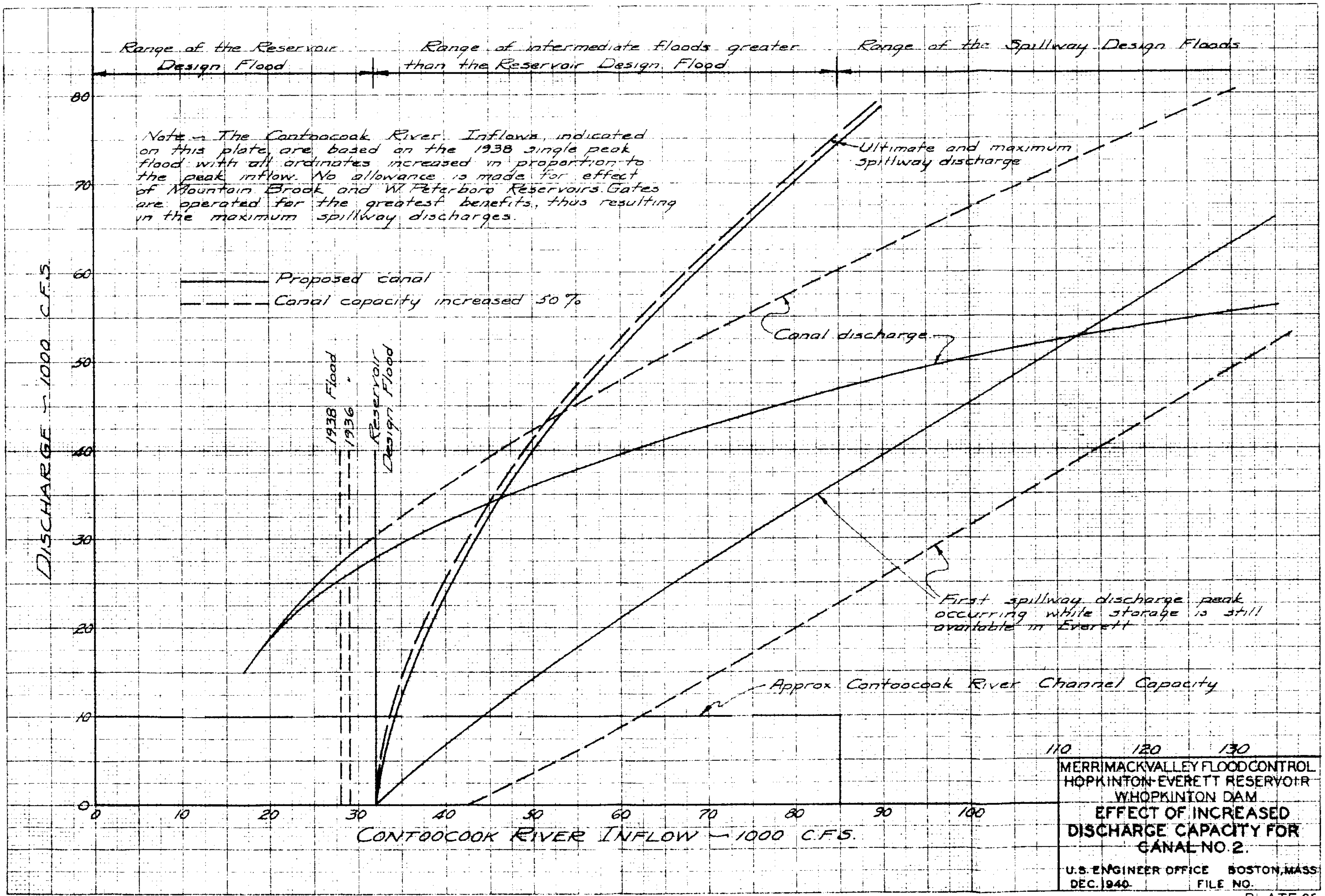

Range of the Reservoir Design Flood
Range of intermediate floods greater than the Reservoir Design Flood
Range of the Spillway Design Floods
Note – The Contoocook River Inflows indicated on this plate are based on the 1938 single peak flood with all ordinates increased in proportion to the peak inflow. No allowance is made for effect of Mountain Brook and W. Peterboro Reservoirs. Gates are operated for the greatest benefits, thus resulting in the maximum spillway discharges.
Ultimate and maximum spillway discharge
Proposed canal
Canal capacity increased 30%
Canal discharge
1938 Flood
1936 "
Reservoir Design Flood
First spillway discharge peak occurring while storage is still available in Everett
Approx. Contoocook River Channel Capacity
DISCHARGE – 1000 C.F.S.
CONTOOCOOK RIVER INFLOW – 1000 C.F.S.
0
10
20
30
40
50
60
70
80
90
100
110
120
130
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
W. HOPKINTON DAM
EFFECT OF INCREASED DISCHARGE CAPACITY FOR CANAL NO. 2.
U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940 FILE NO.

PLATE 65

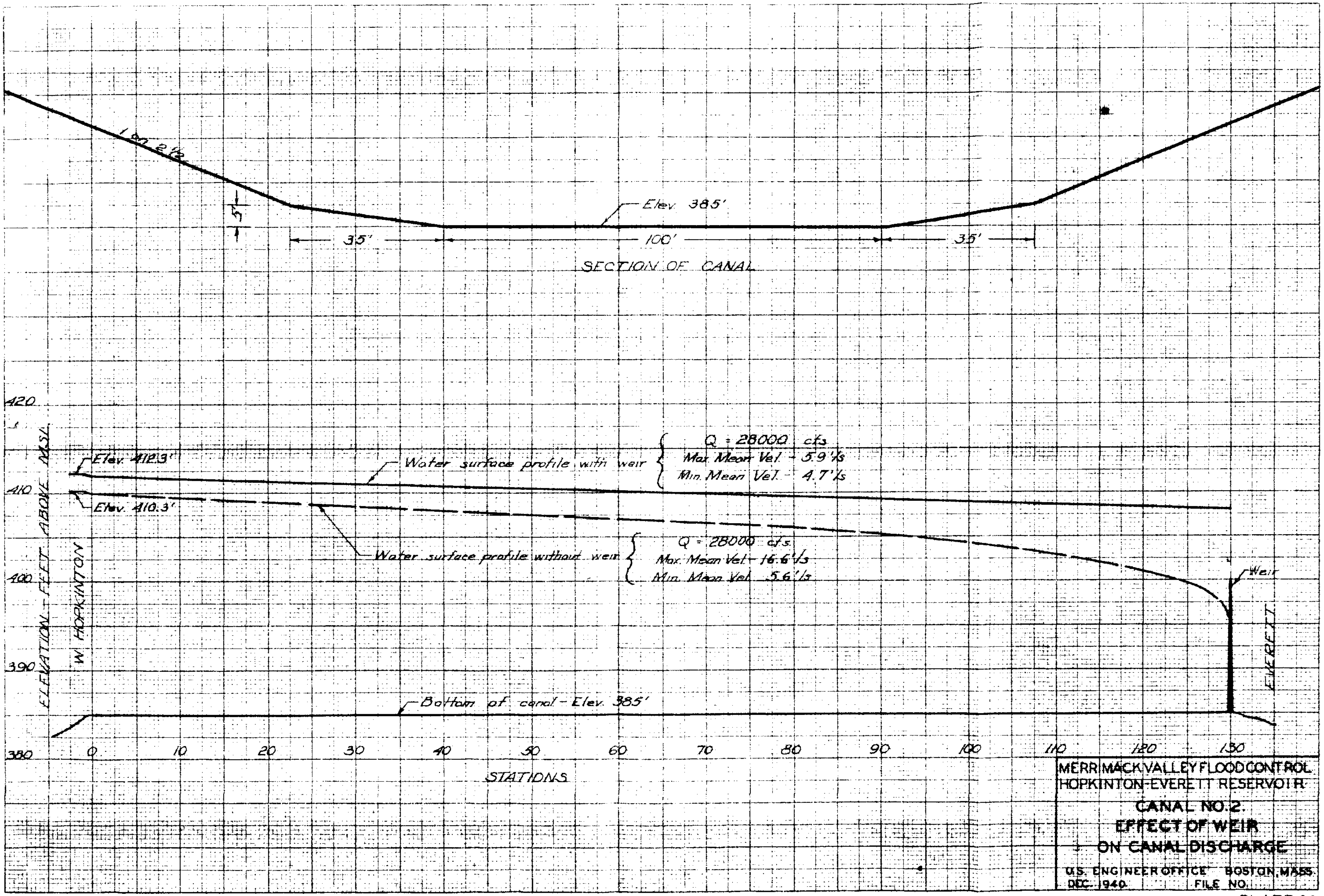

1 on 2½
5'
Elev. 385'
35'
100'
35'
SECTION OF CANAL
420
410
400
390
380
ELEVATION - FEET ABOVE M.S.L.
W. HOPKINTON
EVERETT
Elev. 412.3'
Elev. 410.3'
Water surface profile with weir
Q = 28000 cfs
Max. Mean Vel. - 5.9'/s
Min. Mean Vel. - 4.7'/s
Water surface profile without weir
Q = 28000 cfs
Max. Mean Vel. - 16.6'/s
Min. Mean Vel. - 5.6'/s
Weir
Bottom of canal - Elev. 385'
0
10
20
30
40
50
60
70
80
90
100
110
120
130
STATIONS
MERRIMACK VALLEY FLOOD CONTROL
HOPKINTON-EVERETT RESERVOIR
CANAL NO. 2.
EFFECT OF WEIR
ON CANAL DISCHARGE
U.S. ENGINEER OFFICE BOSTON, MASS.
DEC. 1940 FILE NO.

PLATE 66